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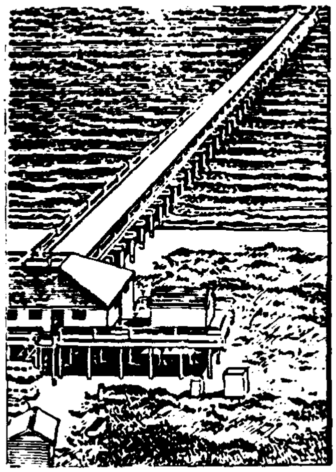
TECHNICAL REPORT CERC-88-1

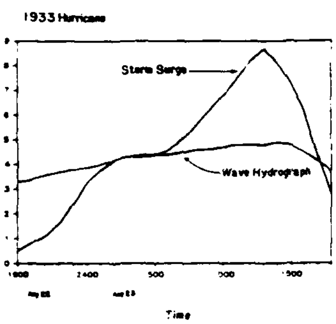
US Army Corps of Engineers

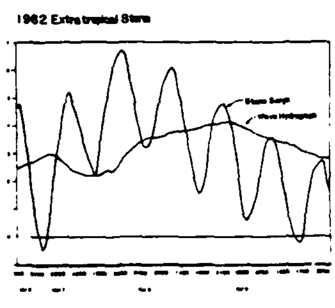
COASTAL ENGINEERING STUDIES IN SUPPORT OF VIRGINIA BEACH, VIRGINIA, BEACH EROSION CONTROL AND HURRICANE PROTECTION PROJECT

Report 2

SEAWALL OVERTOPPING EVALUATION







by

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A study was conducted to determine overtopping rates for a step-faced seawall with curved parapet. The seawall was designed as part of a beach erosion control and hurricane protection project along approximately 6 miles of Virginia Beach, Virginia. Storm damages to the area have included loss of beach, destruction of bulkhead and seawall systems, damage to buildings, and inshore flooding along the commercially developed and urban shoreline.

Using the Storm Time-History Method developed for this study to calculate overtopping rates from results of physical model tests, results show that the design will reduce overtopping to a suitable level. The seawall design consists of a seawall with crest elevation of +15.7 ft NGVD fronted by a beach with elevation +3.4 ft NGVD which was tested using storm surge hydrographs from an August 1933 hurricane and a March 1962 extratropical storm.

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PREFACE

The US Army Engineer District, Norfolk (NAO), requested the US Army Engineer Waterways Experiment Station's (WES's) Coastal Engineering Research Center (CERC) to assist in the design of a Beach Erosion Control and Hurricane Protection Project for Virginia Beach, Virginia. The study was divided into two major parts consisting of a seawall design and a beach nourishment design. This report is the second in a three-report series and addresses the physical model seawall overtopping evaluation. Funding authorizations by NAO were granted in accordance with Intra-Army Order No. AD-86-3018.

This study was conducted at CERC under general direction of Dr. J. R. Houston, Chief, and Mr. C. C. Calhoun, Jr., Assistant Chief, CERC; and under direct supervision of Mr. T. W. Richardson, Chief, Engineering Development Division (CD); Mr. C. E. Chatham, Chief, Wave Dynamics Division (CW); Ms. J. Pope, Chief, Coastal Structures and Evaluation Branch, CD; and Mr. D. D. Davidson, Chief, Wave Research Branch, CW. This report was prepared by Mr. W. J. Lillycrop, Ms. J. Pope, and Dr. C. E. Abel and edited by Ms. S. A. J. Hanshaw, Information Technology Laboratory, WES.

During this study close coordination was maintained through Mr. D. Pezza, NAO Project Manager, and Ms. J. Pope, CERC Project Manager. Acknowledgment is made to all others involved at NAO for their assistance in the study.

The authors extend special acknowledgment to Dr. Charles E. Abel, our friend, colleague, and co-author, who produced all hurricane and extratropical wave simulations and provided valuable guidance and insight. Dr. Abel passed away on 19 April 1987.

Commander and Director of WES during the investigation, preparation, and publication of this report was COL Dwayne G. Lee, EN. Technical Director was Dr. Robert W. Whalin.

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CONVERSION FACTORS NON-SI to SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
cubic feet	0.02831685	cubic metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
miles (US nautical)	1.852	kilometres

COASTAL ENGINEERING STUDIES IN SUPPORT OF VIRGINIA BEACH, VIRGINIA BEACH EROSION CONTROL AND HURRICANE PROTECTION PROJECT

SEAWALL OVERTOPPING EVALUATION

PART I: INTRODUCTION

Project Description

1. The proposed Virginia Beach, Virginia, Beach Erosion Control and Hurricane Protection Project is one of the largest and most complex coastal projects of this type in recent Corps of Engineers experience. The City of Virginia Beach is located on the east coast of the United States just south of the entrance to Chesapeake Bay. The project area consists of 6 miles* of heavily developed commercial and urban shoreline which extends north from Rudee Inlet to 89th Street (Figure 1). This shoreline is subject to severe damages from both hurricanes and extreme extratropical storms as evidenced by the August 1933 hurricane and the March 1962 extratropical storm ("the Ash Wednesday storm") which devastated this coastal area. Storm damages have included loss of the beach, destruction of the bulkhead and seawall system, damage to buildings, and inshore flooding. In addition, there has been a continuing problem with beach erosion. Since 1962 annual harbor dredging of Rudee Inlet and pumping operations to bypass sand at Rudee Inlet, and/or the trucking in of sand from other sources have been sponsored by the Federal, state, and city governments to maintain a beach width of approximately 100 ft with a crest elevation of +5.4 ft.

2. Existing protection consists of a combination of various bulkheads with crest elevations between 10 and 12 ft National Geodetic Vertical Datum (NGVD) and nourished beach. In 1970 the US Army Engineer District, Norfolk (NAO), completed a feasibility study which recommended construction of a sheet-pile seawall with a concrete cap at elevation 15 and heavy stone at the base. By 1983, results of the previous study had been reevaluated and incorporated into an initial (Phase I) seawall design and beach erosion control

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* A table of factors for converting non-SI to SI (metric) units of measurement is presented on page 3.

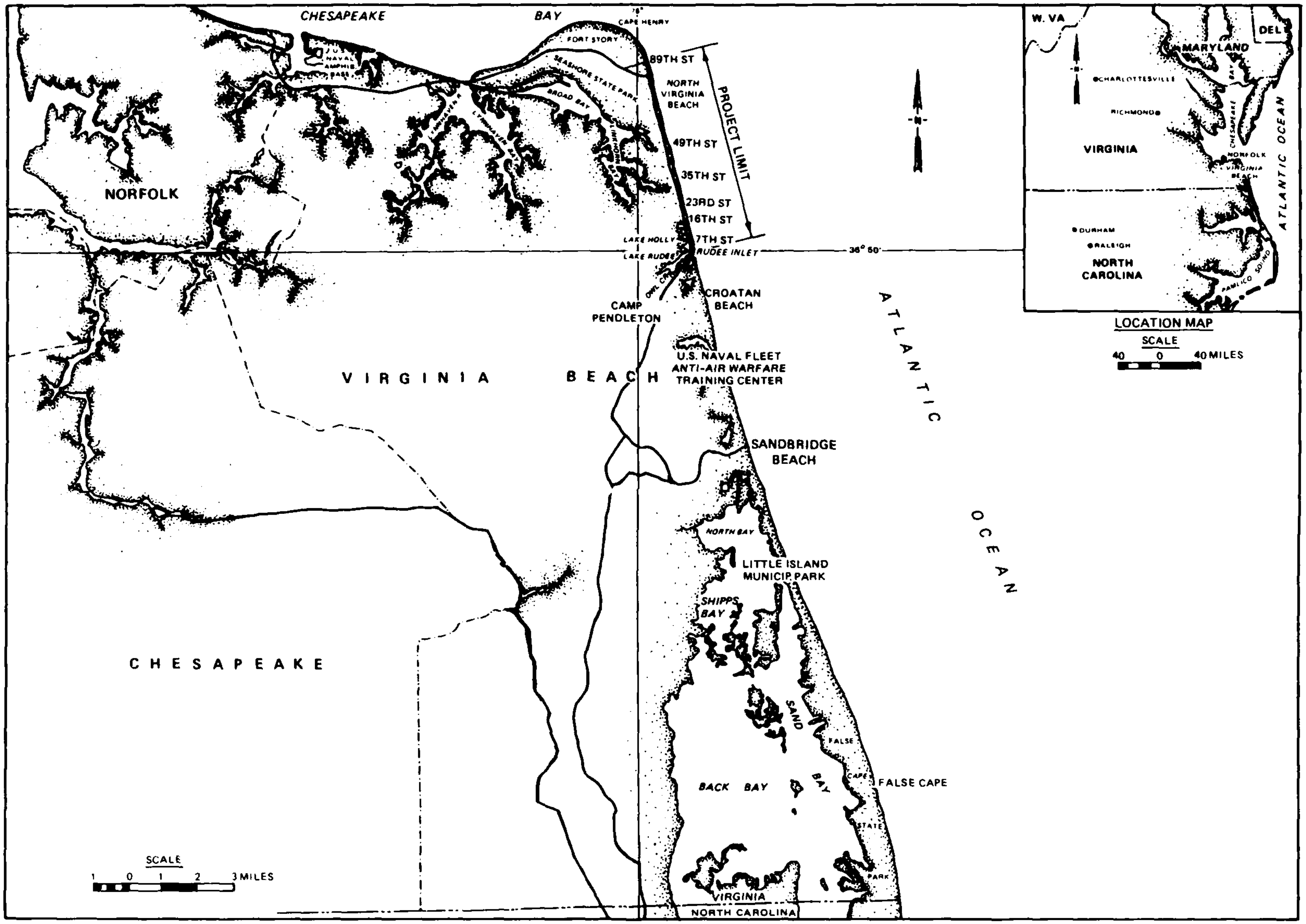


Figure 1. Location map

concept. The seawall was designed with guidance from the Shore Protection Manual (SPM) (1984) which is based primarily on monochromatic wave theory. Adequate storm protection was to be provided by the seawall without sacrificing aesthetics of the ocean view.

3. The proposed plan consists of constructing a new stepped-face seawall with curved parapet located just seaward of the existing seawall (between Rudee Inlet and 57th Street). The existing dune field will be raised and widened as necessary from 57th Street, north to 89th Street. Both structures would be fronted by a continuously maintained beach berm.

Study Background

4. This report is second in a series of three reports on coastal engineering studies conducted by the US Army Engineer Waterways Experiment Station's Coastal Engineering Research Center (CERC) to assist NAO in advanced engineering and design of the Virginia Beach, Virginia, Beach Erosion Control and Hurricane Protection Project. The other two reports discuss physical model studies and the beach and dune design. The overall study is divided into two major sections consisting of the seawall design (i.e., physical model, overtopping tests, and physical model pressure or wave loading tests) and the beach and dune design evaluation. Figure 2 is a schematic presentation of the coastal engineering studies. This report presents selection of the design parameters and an analysis of the results of the physical model seawall overtopping tests (items 10-19). The other two reports deal with the actual physical model tests for overtopping and measurements for wave loading (items 15 and 20) and the beach and dune design (items 21-31).

5. Selection of design waves, storm surge hydrographs, and runup-overtopping rates was crucial to developing the most hydraulically efficient seawall geometry and in analyzing short-term beach stability. Coastal engineering studies in support of the seawall design consisted of selecting design storms from the historical record, simulating the wave field for each of these storms, establishing the design surge hydrographs, and developing a two-dimensional hydrographic model to predict overtopping rates.

6. Seawall overtopping tests involved hydraulically designing the most efficient seawall plan and developing overtopping rates which could be used in interior flooding design. The study used a two-dimensional physical model

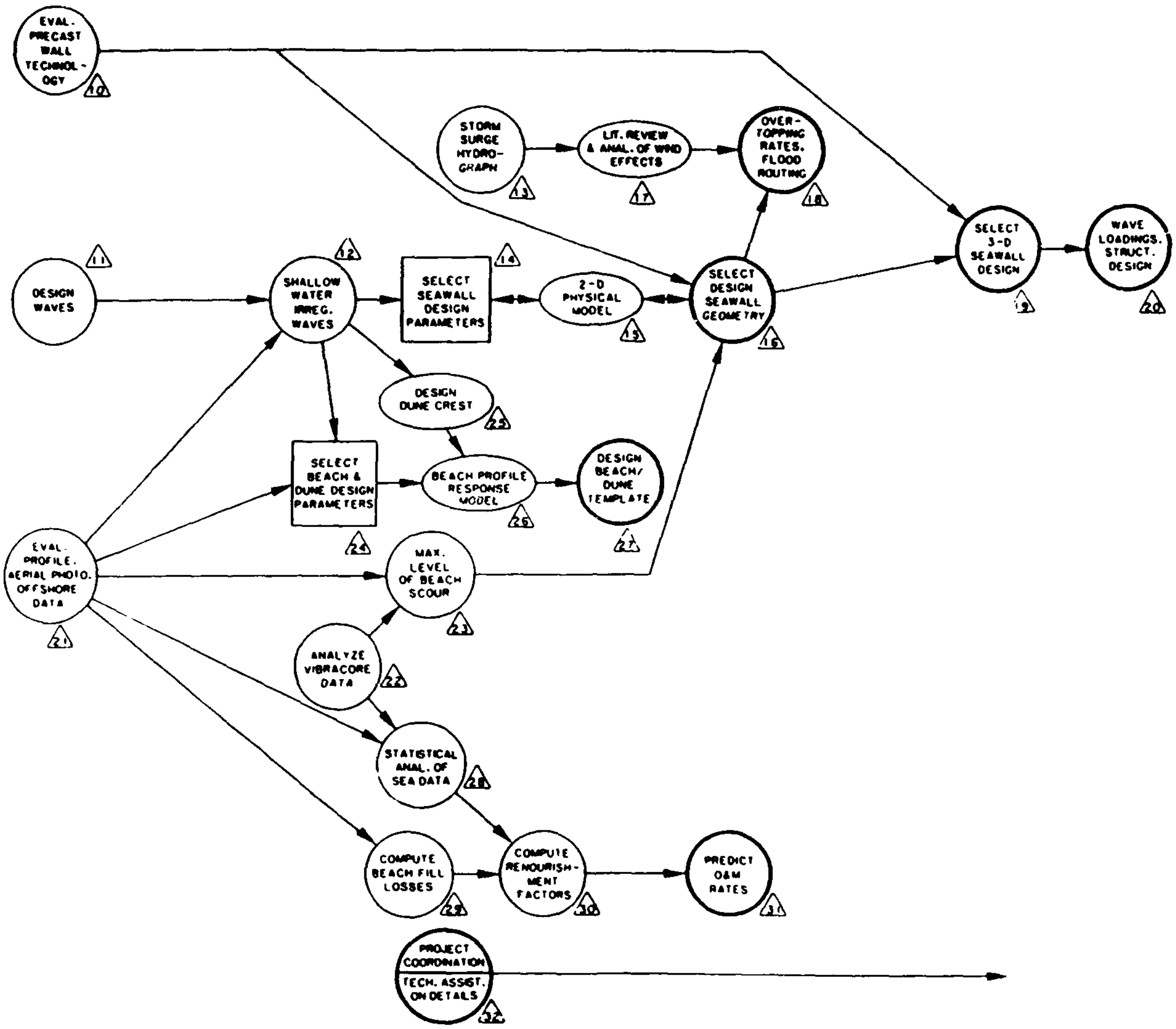


Figure 2. Flow chart for coastal engineering studies

(Heimbaugh et al. 1988) over a range of sea state conditions to measure wave-induced overtopping. Parameters which required evaluation and were incorporated into the model test were the design wave conditions, the design storm surge levels, the seawall geometry, and the beach profile.

7. Model tests were run using spectral waves on two different seawall cross sections or templates. A Phase I template resulted in a peak overtopping rate in excess of the value predicted of 0.125 cfs/ft of seawall in a general design memorandum (NAO 1983). Selected modifications were made to the wall cross section to reduce these quantities, and the Phase II wall did lower the overtopping rates (Heimbaugh et al. 1988).

8. This report discusses the selection of the design parameters, summarizes the results of the physical model tests, presents the analysis tools used for data evaluation, and interprets the model results to compute overtopping hydrographs.

PART II: STORM SURGE HYDROGRAPHS

9. Design water level conditions were provided by NAO and selected from a review of historic storm events that have impacted the Virginia Beach area. Eleven storms of record are ranked by peak storm surge height, and the maximum three for each class of storm are selected (Table 1). Water level elevations shown are for a tide gage (Norfolk Harbor) located approximately 10 miles inside the Chesapeake Bay. These data were used because no gage was located at the study site during the storm event or the storm or because the gage failed or was destroyed. Select data were later corrected by NAO for use at the study site. The three severest storms (highest ranking) of each class (i.e., either a hurricane or extratropical) were evaluated to determine which design conditions would be used in physical model tests.

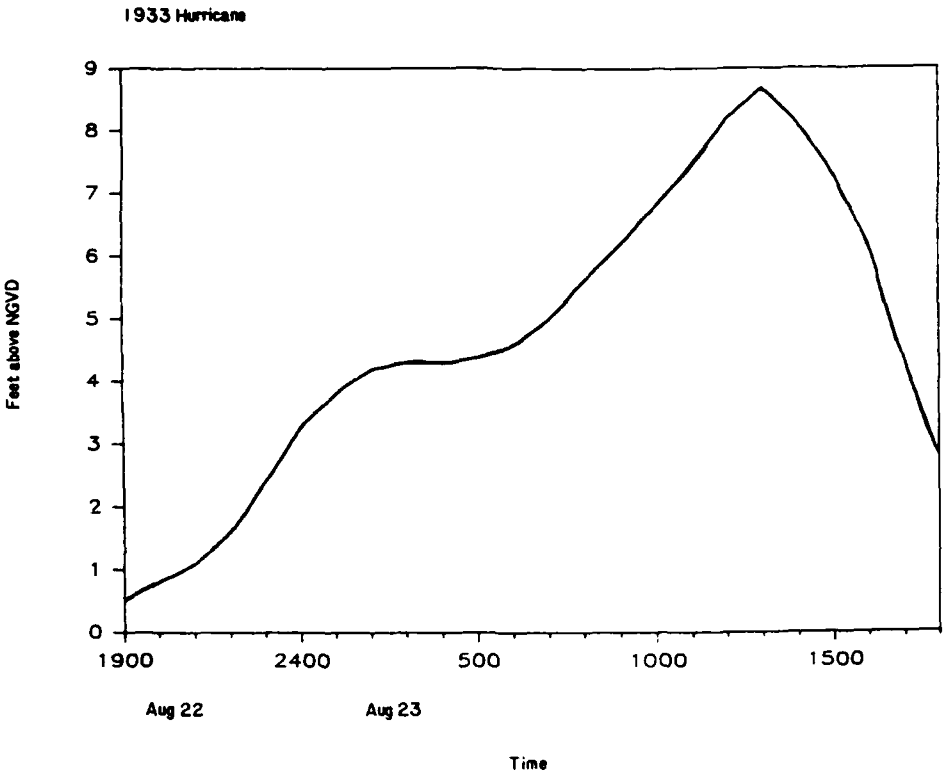
Table 1

Historic Storm Events

Date	Class of Storm*	Surge Height ft	Storm Ranking Hurricane	Storm Ranking Extratropical
23 Aug 1933	H	8.05	1	
18 Sep 1936	H	7.55	2	
7 Mar 1962	E	7.06		1
16 Sep 1933	H	6.35	3	
11 Apr 1956	E	6.34		2
12 Sep 1960	H	6.09	4	
18 Sep 1928	H	5.85	5	
27 Apr 1978	E	5.84		3
27 Sep 1956	H-E**	5.74		4
6 Oct 1957	E	5.53		5
5 Oct 1948	E	5.35		6

* H - hurricane; E - extratropical storm.
** Before impacting the study area, this hurricane was reclassified as an extratropical storm.

10. The evaluation considered the three major hurricanes and three major extratropical storms of record. Documented wind fields associated with the six events were then used in the numerical model SHALWV (Hughes and Jensen 1986) to hindcast the deepwater waves and to transform them into shallow water. From this analysis the severest storm event for each class of storm, hurricane and extratropical, was used in the model tests. Figure 3



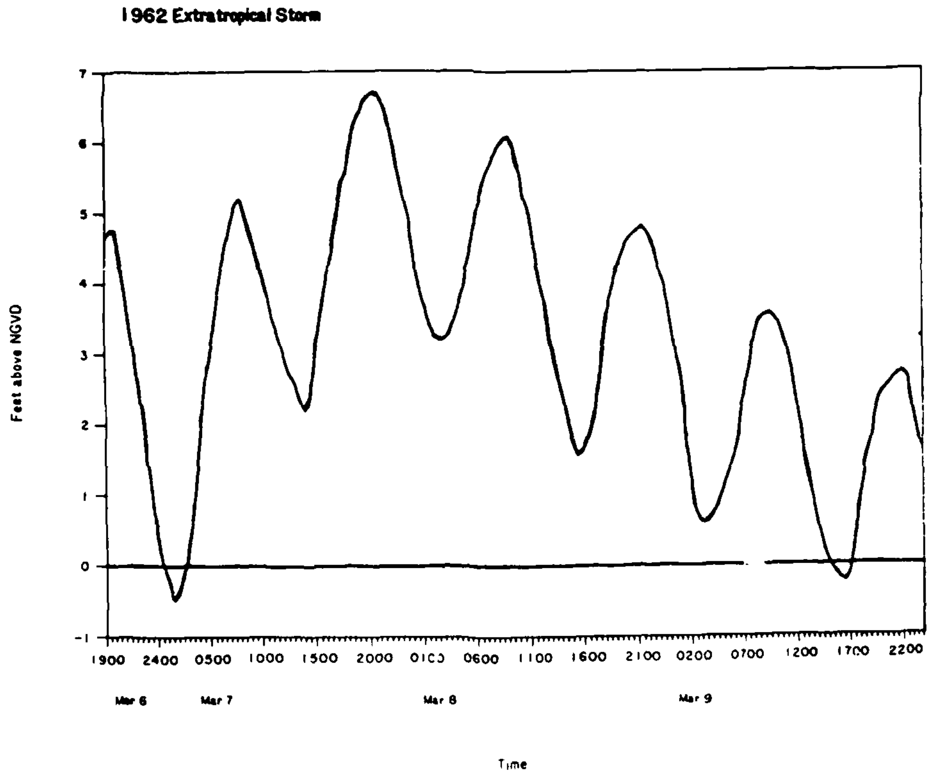


Figure 3. Observed tide and storm surge hydrographs of August 1933 hurricane and March 1962 extratropical storm

illustrates the storm surgc hydrographs for these two storms, the 23 August 1933 hurricane and the 7 March 1962 extratropical event.

11. Because the approach used to choose design storm conditions was deterministic, attaching a frequency of occurrence to the overtopping calculations could only be accomplished through the development of an accompanying stage/frequency relationship based on a probabilistic model. However, the selected storms were the worst of record (record length is 50 years).

PART III: STORM WAVE HINDCASTS

12. Wind wave spectra for hurricane and extratropical storms affecting the Virginia Beach, Virginia, area during the periods 1928 to 1978 were calculated. Data recorded in the vicinity of Norfolk, Virginia, from five hurricanes and six extratropical storms were used for storm selection and ranked by peak still-water level (swl) (Table 1). The three highest-ranked hurricanes and the three highest-ranked extratropical storms were selected for wave hindcasting using numerical simulation. Hindcast periods which were modeled for each of the six storms are listed in Table 2.

Table 2

Selected Storms

Swl Rank	Hindcast Period*
	Hurricane
1	21 Aug 1933 0000Z -- 24 Aug 1933 0000Z
2	14 Sep 1933 0000Z -- 17 Sep 1933 0000Z
3	15 Sep 1936 1200Z -- 19 Sep 1936 0000Z
	Extratropical
1	6 Mar 1962 0000Z -- 9 Mar 1962 1200Z
2	11 Apr 1956 2100Z -- 14 Apr 1956 1800Z
3	26 Apr 1978 0000Z -- 29 Apr 1978 0000Z

* Z = Greenwich mean time.

13. Three different modeling procedures were required to perform the hindcasts based on type of storm and available data.

a. All hurricane wind fields were computed by a planetary boundary layer (PBL) vortex model.* Necessary input data for the PBL model were obtained from the National Weather Service's tropical cyclone data set TD-9697. Computed winds were then used as forcing functions for the wind wave spectral transformation model.

* Unpublished report by V. J. Cardone (President, Oceanweather) et al. (1981), titled "Unified Program for the Specification of Hurricane Boundary Layer Winds Over Surfaces of Specified Roughness," Unpublished Report for US Army Engineer Waterways Experiment Station, Vicksburg, MS.

b. Wind and wave hindcast data for the extratropical storms of April 1956 and March 1962 were available from the Coastal Engineering Research Center's Sea State Engineering and Analysis System (SEAS) (McAneny 1986). Appropriate values of wind and wave parameters were extracted from the SEAS data base. These estimates were then transformed into boundary forcing functions for the wave model.

c. The extratropical storm of April 1978 is too recent for inclusion in the SEAS data base which currently spans the period 1956 to 1975. Therefore, a supplementary analysis was required. Surface pressure maps were digitized, and wind fields were computed using the atmospheric boundary layer model developed for CERC's Wave Information Studies (WIS). Previous applications of the WIS wind model form the basis of the hindcast results available in SEAS. The computed winds were applied as forcing functions to the wave spectral transformation model.

14. All wind wave computations were made with the WIS discrete spectral wave growth transformation model (Hughes and Jensen 1986). This model predicts the frequency and directional transformation of forced wind waves in deep, intermediate, and shallow water. A sequence of three nested, coupled computational grids was used to bring the hindcast results to a point approximately 12 nautical miles east of the Virginia Beach area, at longitude 75°45'W and latitude 36°50'N. Mean water depth at this location is 11 m. The frequency spectra of sea surface variance for the times of maximum wave height at this location are depicted in Figure 4 for both hurricanes and extratropical storms. A summary of the hindcast peak wave characteristics is in Table 3.

15. The hindcast for the April 1978 storm was compared to gage measurements at CERC's Field Research Facility, at Duck, North Carolina. The range of wave heights and periods hindcast compared well with gage data in about 10 m of water.

16. Of the hurricanes, the August 1933 storm was by far the most severe due to its passage in an onshore direction just to the south of the study area. The storm produced the highest swl of record (record length being 50 to 60 years) with a maximum projected surge of +8.7 NGVD for Virginia Beach and the highest significant wave. The March 1962 extratropical storm is one of the most severe storms of this class to affect the mid-Atlantic coast. Wave heights of 10 to 12 m were hindcast in deep water beyond the continental shelf. The largest component of the waves was high energy swell propagating across the shelf to Virginia Beach. For these reasons the August 1933

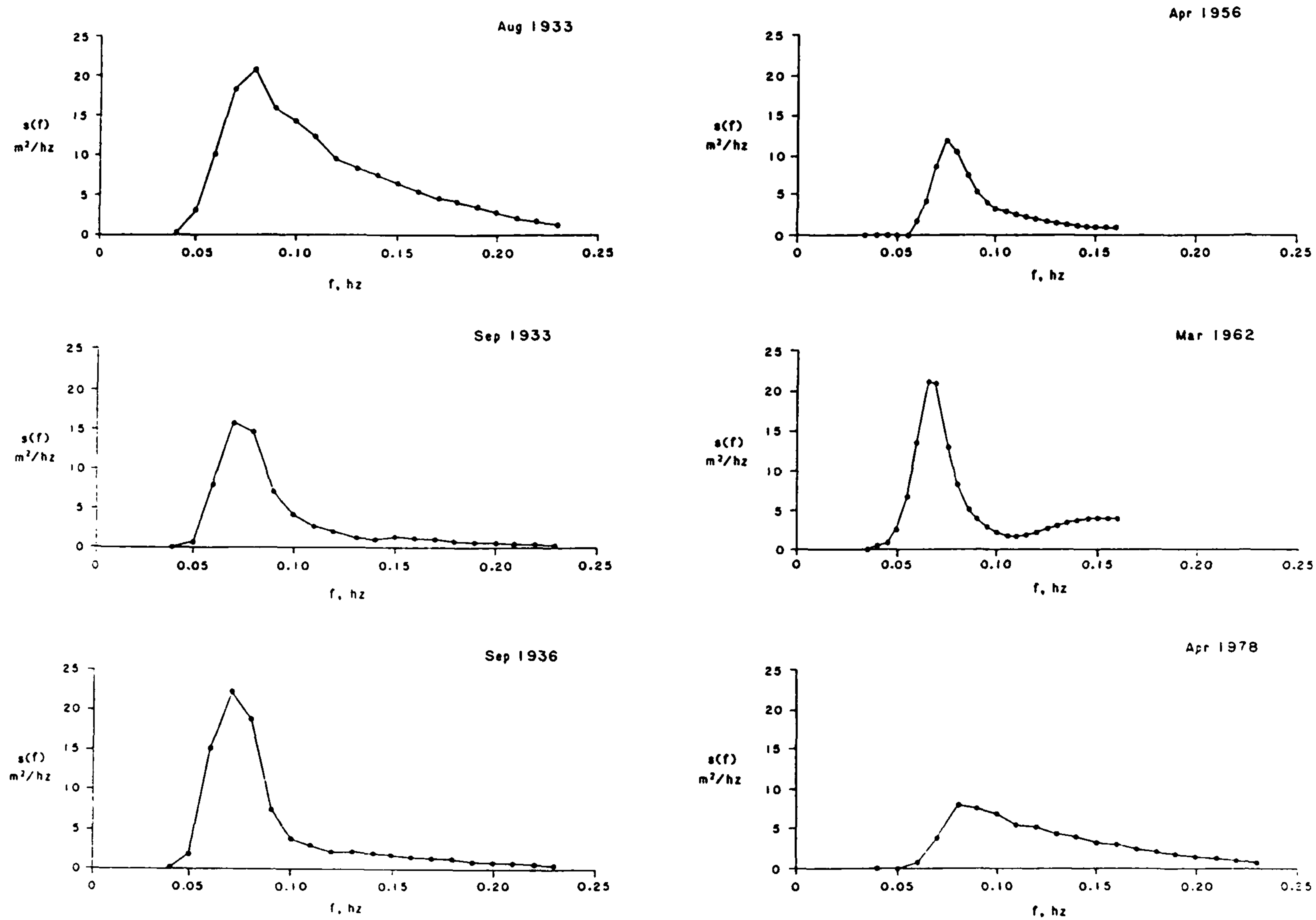


Figure 4. Frequency spectra of sea surface variance

Table 3

Hindcast Summary

Date	H_{mo} m	Peak Period sec
	Hurricane	
Aug 1933	4.82	13.7
Sep 1933	3.28	14.3
Sep 1936	3.75	14.3
	Extratropical	
Apr 1956	3.08	13.3
Mar 1962	4.14	15.4
Apr 1978	3.39	12.2

hurricane and the March 1962 extratropical storm wave spectra were approved by NAO as the design conditions for the physical model tests.

17. The hindcast frequency spectra of sea surface variance were computed at 26 discrete frequencies for the March 1962 extratropical storm and at 20 discrete frequencies for the August 1933 hurricane. Since these spectra were to be the input to a wave generator in a flume, a higher resolution representation was required. This representation was provided by fitting the hindcast spectra to the TMA spectral shape function (Hughes 1984). TMA is an analytical spectrum representing the depth and frequency dependent transformations of a deepwater wave moving into shallow water. Local water depth, significant wave height, peak period, and a set of three spectral shape parameters allow for the evaluation of the spectrum at any desired frequency and at any desired location shoreward of the initial computational site. The spectra and these results were applied to both the physical model and the beach and dune studies for the Virginia Beach project.

PART IV: SEAWALL GEOMETRY

Precast Wall Technology

18. Prior to model studies, precast seawall technology was reviewed to determine if any particular device complying with such a construction methodology could be employed in the Virginia Beach seawall design. A literature review, along with establishment of several industry contacts with companies involved in constructing precast coastal structures and discussions with several researchers in other nations, was conducted. A few patented precast devices (such as stresswall by Stresswall International and Neptune Caisson by Mitsubishi Corp.) were identified as possibly applicable and pursued through more detailed discussions. Reinforced Earth, a recent seawall installation at Pacifica, California, was inspected. However, none of the patented devices could be both constructed in the stepped-wall configuration required for Virginia Beach and demonstrated to be stable in a high wave energy coastal environment.

19. Although it could be feasible to develop a precast unit or plan which could be used to construct the Virginia Beach seawall components off site, such a plan would need careful structural and hydrodynamic assessment better conducted as an independent detailed structural design. The scope of this task and time constraints associated with the overall study did not allow for development of the specific structural component. The proposed seawall configuration was therefore not adjusted to incorporate any existing precast system. Development of a precast system tailored for use at Virginia Beach may be a subject for consideration during later phases of the design or as a value engineering activity.

Physical Model Tests

20. The physical model tests were run in two phases. Phase I was an initial effort consisting of observations of the seawall and wave response and the stability of the stone toe in addition to wave gage data and overtopping measurements. Phase II of the model tests incorporated changes to the seawall design based on the Phase I results. A summary of the model tests is provided below. Heimbaugh et al. (1988) contains a detailed report on these tests.

21. The design cross section of the Phase I model tests is presented in Figure 5 with key elements which include a stepped seawall with parapet face designed for reducing wave overtopping by reflecting incident waves in a seaward direction. The design crest elevation was 15.7 ft above NGVD. As designed for the model study, the seawall would be supported on vertical sheet piles with a stone toe berm consisting of riprap. The riprap had a design elevation of 3.4 ft above NGVD and a design width of approximately 5 ft. The face of the riprap had a 1V:2H slope down to +1.0 NGVD. The proposed beach berm, which was not incorporated in the model tests, would then be placed over the toe berm to an elevation of 5.4 ft above NGVD.

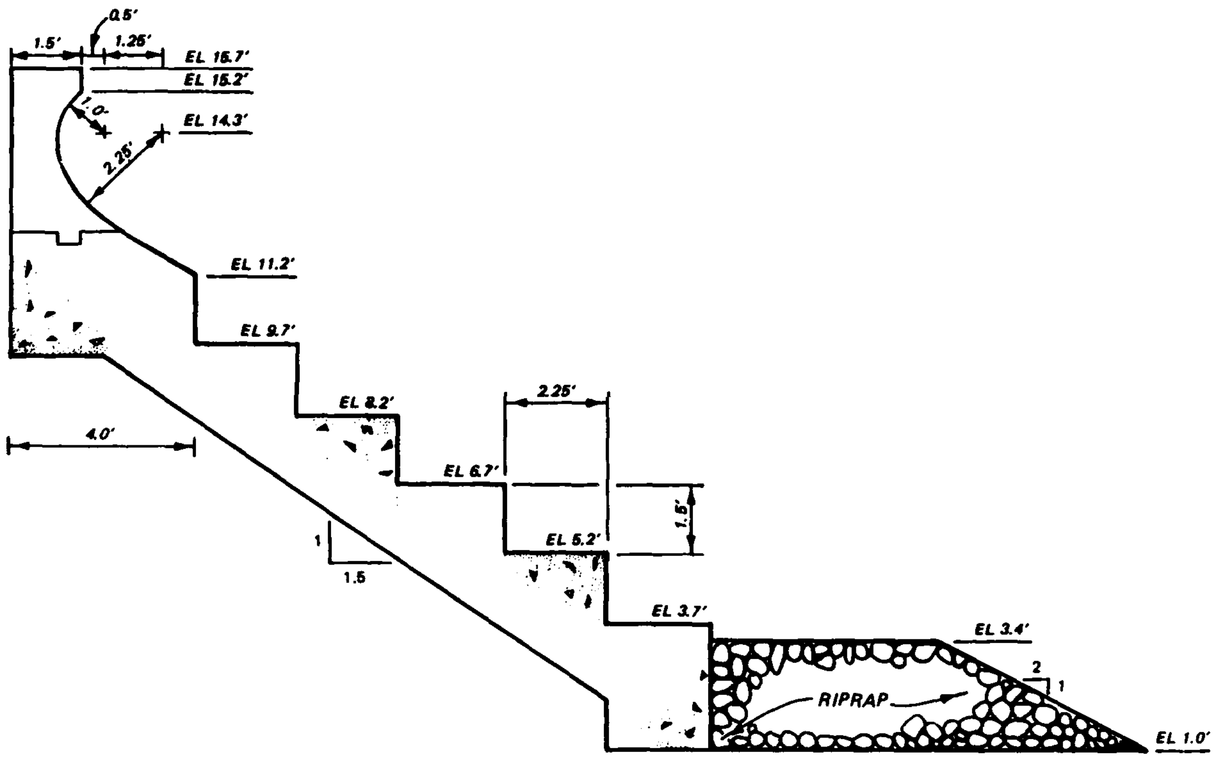


Figure 5. Phase I cross section

22. Selection of this design profile fronting the seawall required assessment of the maximum storm-induced scour which could be expected to accompany either of the design storms. Phase I GDM calculations were based on scour to a depth of +3.4 ft NGVD or approximately 2 ft of erosion. Virginia Beach core borings from 1948 and 1968 (pre- and post-1962) were reviewed by NAO personnel who noted that the 1962 extratropical storm eroded the general beach elevation to approximately 1 ft below NGVD. However, there was no

information available on the condition of the beach immediately prior to this storm.

23. A review of existing literature on seawalls and their effect on beaches has recently been conducted at CERC (Kraus, in press). Several laboratory and field studies have suggested a rule-of-thumb estimate of the depth of scour in front of a seawall equal to the height of the incident wave which could be supported by the water depth (approximately 2 ft in this case). The physical model tests were set up with eroded beach elevation at +1.0 ft NGVD during the Phase I tests to allow for testing of the stability of the stone berm under a severely eroded beach condition and kept at that elevation for the Phase II tests.

24. A total of 110 tests was made for the Phase I template with four water levels of 6.0, 7.0, 8.0, and 9.5 above NGVD for the two design storms discussed in Part III of this report. The Phase I results were higher than those predicted in the Phase I GDM study (i.e., 0.125). In an attempt to reduce the overtopping rates, two modifications to the Phase I testing plan were considered: modifying the stone toe berm and modifying the seawall. The stone toe berm was proposed to control overtopping by controlling scour during overtopping events. The width of the berm (three stones wide) was determined in Phase I testing to be stable. A wider toe would be more conservative but felt to be unnecessary and would have little influence on overtopping unless the width was increased substantially. Modifications to the seawall included adding a 0.75-ft wider lip at the top of the curved parapet. The addition of this lip required a slight redesign of the curved parapet and steps.

25. The Phase I seawall was constructed at a scale of 1:13 which, for the wave tank used, allowed only 60 to 70 percent of the maximum H_{mo} to be produced at the wave board. Although the tank wave gages suggested that shallow-water wave heights were not significantly increasing with increasing deepwater wave heights, it could not be determined whether the shallow-water wave height had reached its maximum. Thus, to assure the design events could be reproduced, the Phase II tests were conducted at a 1:19 scale which allowed 100 percent of the design H_{mo} spectra to be produced.

26. The Phase II seawall template was constructed at the smaller scale and installed in the same tank for testing. Figure 6 shows the modified seawall design and test facility cross section. A foreshore slope of 1 on 16 and offshore slope of 1 on 100 were used in the model tests. A total of 155

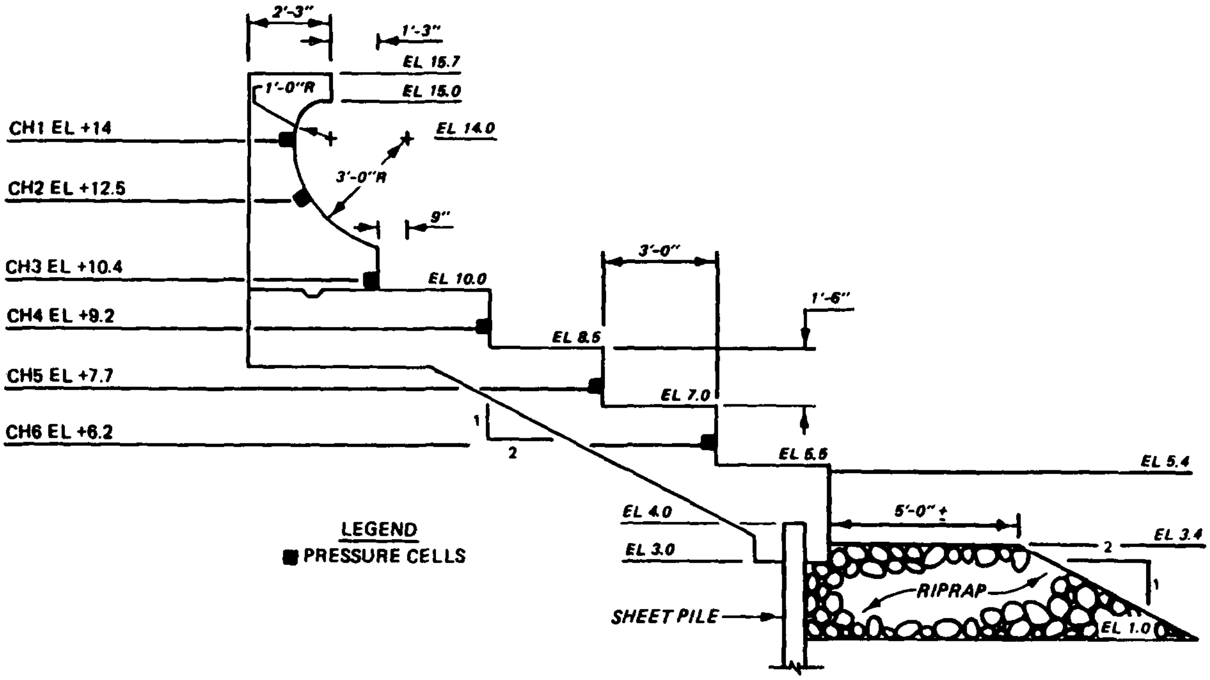


Figure 6. Phase II cross section

overtopping tests was run. Variables tested included three water levels at 7.0, 8.0, and 9.5 ft swl above NGVD for the two design wave spectra (hurricane and extratropical storm). The lower range swl was raised to 7.0 ft above NGVD because there was no overtopping at 6.0 ft during the Phase I tests.

27. Appendix A presents a summary of the Phase II tests. The percent gain referred to in column 3 represents the change in deepwater wave height based on an increase or decrease in wave height at the wave board. Various percentages of the design wave spectra were modulated for deepwater H_{mo} from 30 to 100 percent.

PART V: ANALYSIS

28. Both the maximum overtopping rate and the total overtopped volume are important in evaluating damages associated with storms. Determining the rate (a function of wave height and swl) and the volume (a function of the overtopping rate through time) is a necessary exercise for evaluating potential damages and developing a flood routing plan. The data base collected during the physical model study represents overtopping rates for a relatively narrow set of conditions. Therefore, to evaluate overtopping associated with various design situations, two methods which extend the physical model data base were used: the Storm Time-History Method (STHM) and the Relative Freeboard Method (RFM).

Storm Time-History Method

29. The STHM groups the physical model study data into a series of smaller data sets which are then used to predict overtopping rates throughout the storm as surge and wave conditions evolve. Overtopping tests (presented in Appendix A) were separated into subsets based on storm type (hurricane or extratropical) and by percent of wave height (or gain) produced at the wave paddle in 10 percent increments, essentially separating the storm wave heights by a percent of the fully developed storm conditions.

30. Subsets, which range in number of samples or overtopping values between 8 and 14, are presented in Appendix B. The subset for the hurricane model test with 100 percent gain is shown in Table 4. The swl's are in feet above NGVD, and the overtopping rates are in cubic feet per second per linear foot of seawall. This case will be further developed as an example of the calculations. Only final results from the other percent gains for the extratropical storms and hurricanes will be presented in the main text.

31. Linear regression techniques were employed to produce graphs which relate the overtopping rate to swl for each 10 percent increment of the gain. Regression estimates for overtopping rate were made, and curves were plotted for each storm type and each 10 percent increment of gain. Figure 7 shows the regression curve for the data points in Table 4. The solid line (noted by Q_r) represents the regression estimates with the dashed lines being upper and lower (Q_u and Q_l, respectively) limits for symmetric prediction intervals at a

Table 4

100-95 Percent Gain Hurricane Model Results*

Swl ft	Q** cfs/ft	Swl ft	Q cfs/ft	Swl ft	Q cfs/ft
9.5	0.704	8.0	0.585	7.0	0.148
9.5	1.040	8.0	0.442		
9.5	1.157	8.0	0.480		
9.5	1.058	8.0	0.535		
9.5	0.787	8.0	0.307		
		8.0	0.346		

* For this case, the 100 percent gain data and the 95 percent gain data were combined.

** Q = measured overtopping rates during the model test.

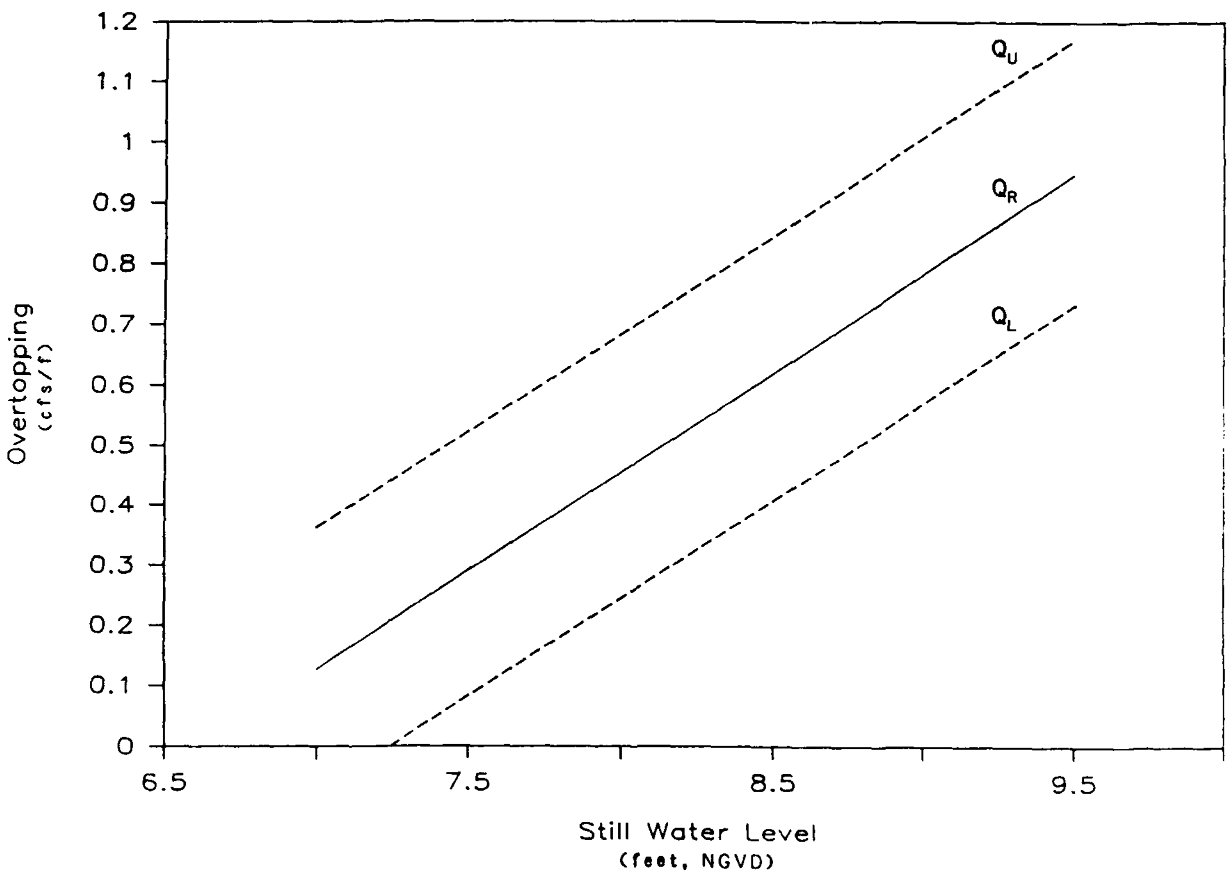


Figure 7. Swl versus overtopping rate

probability level of 90 percent. Notably these limits are not confidence limits on the estimator function but are probability intervals for individual predictions of overtopping for a given swl.

32. A table of overtopping calculations throughout the length of each storm was developed (Table 5). Dates and times are listed in the first two columns with storm surge levels, in feet above NGVD, in the next column. The prototype storm hydrographs were for peak surge levels of +6.7 and +8.7 ft NGVD for the extratropical storm and hurricane at Virginia Beach, respectively. To adjust or scale them to the +7.0-, +8.0-, and +9.5-ft surge levels (swl) used in the physical model tests, linear scaling was applied. These scaled water levels (in feet) are shown in column four. Column five is the deepwater wave height (in metres) calculated for each hour of the storm using the storm wave hindcast model (Part III). The next column is the percent of maximum wave height at that time during the storm. Figure 8 illustrates the time series of the hindcast storm wave hydrograph relative to the measured storm surge hydrograph for both storms.

Table 5

Overtopping Calculations

1933 Hurricane

						Regression Curve Overtopping Rates cfs/ft		
Date	Time	Swl ft	Swl* ft	H_{mo}** m	Percent	Q_r	Q_u	Q_1
Aug 23	0600	4.6	--	4.4	91	0.0	0.0	0.0
	0700	5.0	--	4.5	93	0.0	0.0	0.0
	0800	5.6	5.2	4.6	95	0.0	0.0	0.0
	0900	6.2	5.7	4.6	95	0.0	0.0	0.0
	1000	6.8	6.3	4.7	98	0.0	0.14	0.0
	1100	7.5	6.9	4.8	100	0.09	0.33	0.0
	1200	8.2	7.5	4.8	100	0.30	0.53	0.08
	1300	8.7	8.0	4.8	100	0.45	0.66	0.25
	1400	8.1	7.5	4.8	100	0.27	0.50	0.05
	1500	7.3	6.7	4.8	100	0.03	0.27	0.0
	1600	6.2	5.7	4.5	93	0.0	0.0	0.0
	1700	4.3	--	4.1	85	0.0	0.0	0.0

* Scaled to design storm elevations.

** ${H_{mo}}_{max}$ = 4.82 m.

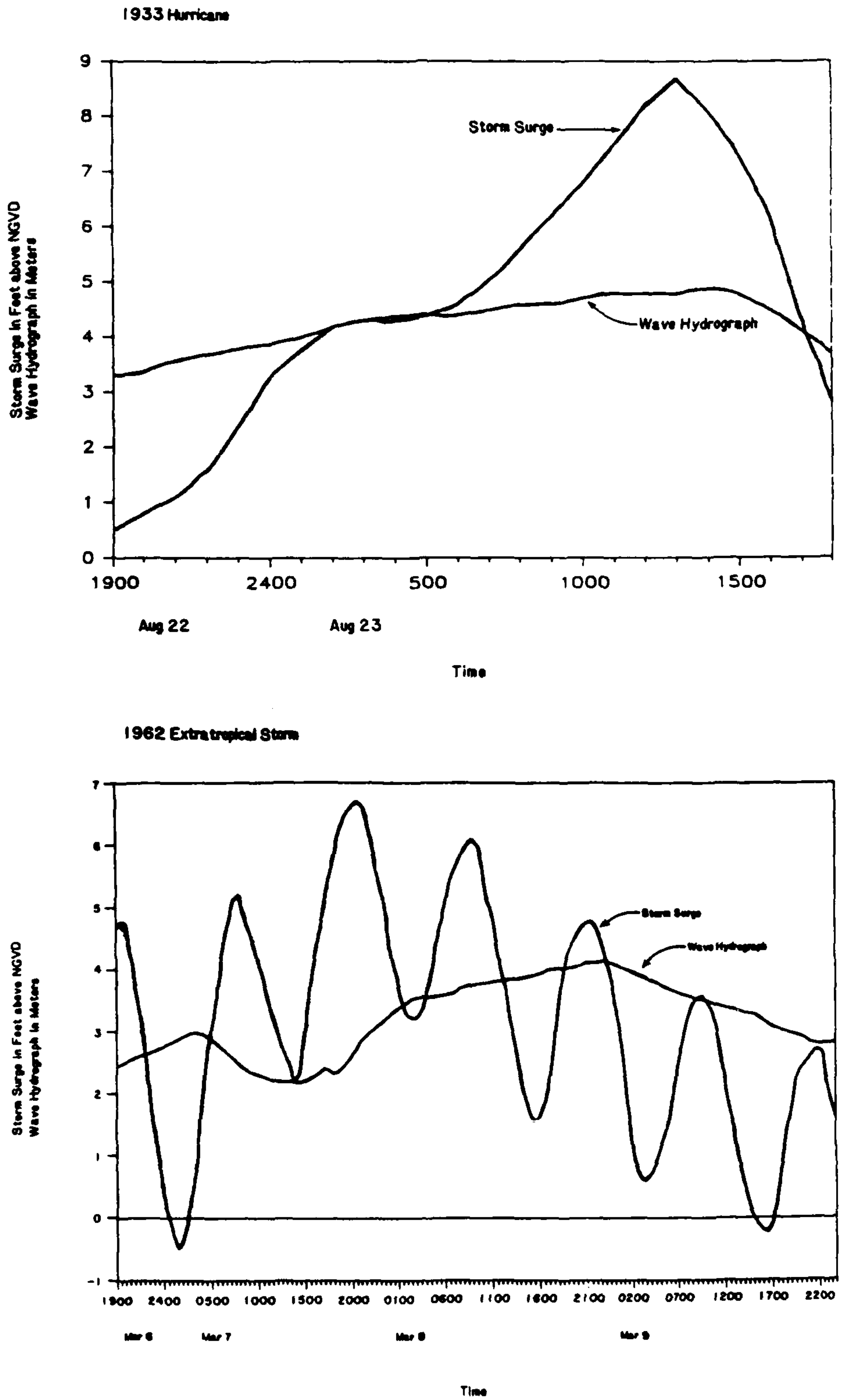


Figure 8. Storm surge hydrograph and storm wave hydrograph

33. Based on these percents of the storm wave hydrograph (Table 5, column 6), the appropriate regression curve was used to obtain the overtopping rate for each hour of the storm. The last three columns in Table 5 are the overtopping rates from the regression curve and the upper and lower prediction intervals, respectively. The values are cubic feet per second per linear foot of seawall and are plotted in Figure 9. Complete results of the study are presented in Part VI.

Relative Freeboard Method

34. The RFM is an expression which was originally developed during the Roughan's Point physical model study (Ahrens, Heimbaugh, and Davidson 1986). Because seawall overtopping rates are dependent on local wave heights and wavelengths, relative freeboard parameter was defined. It is best applied to situations where the shallow-water wave climate is known or has been simulated for the design storm conditions. Relative freeboard is defined as

$$F' = \frac{F}{\left(H_{mos}^2 L_p\right)^{1/3}}$$

where

F = average freeboard, defined as the distance between the crest of the seawall and the local swl

H_{mos} = zero-moment wave height at the toe of the structure

L_p = significant wave length associated with the peak period at the toe of the structure

35. The STHM was developed to supplement the RFM because the RFM merges all test conditions (extratropical storm and hurricane sea conditions with nondesign storm related wave heights) into a single data set to calculate overtopping rates. The two storms include different wave spectra (Figure 4) and related overtopping characteristics. Combining the physical model data did not appear appropriate for computing the design peak overtopping rate.

36. The design data of interest, which occurred at high wave energies, did not fit the general RFM exponential curve based on the entire data base. Figure 10 shows the exponential relation which does not account for the higher data points located near the center of the graph. The deepwater wave height

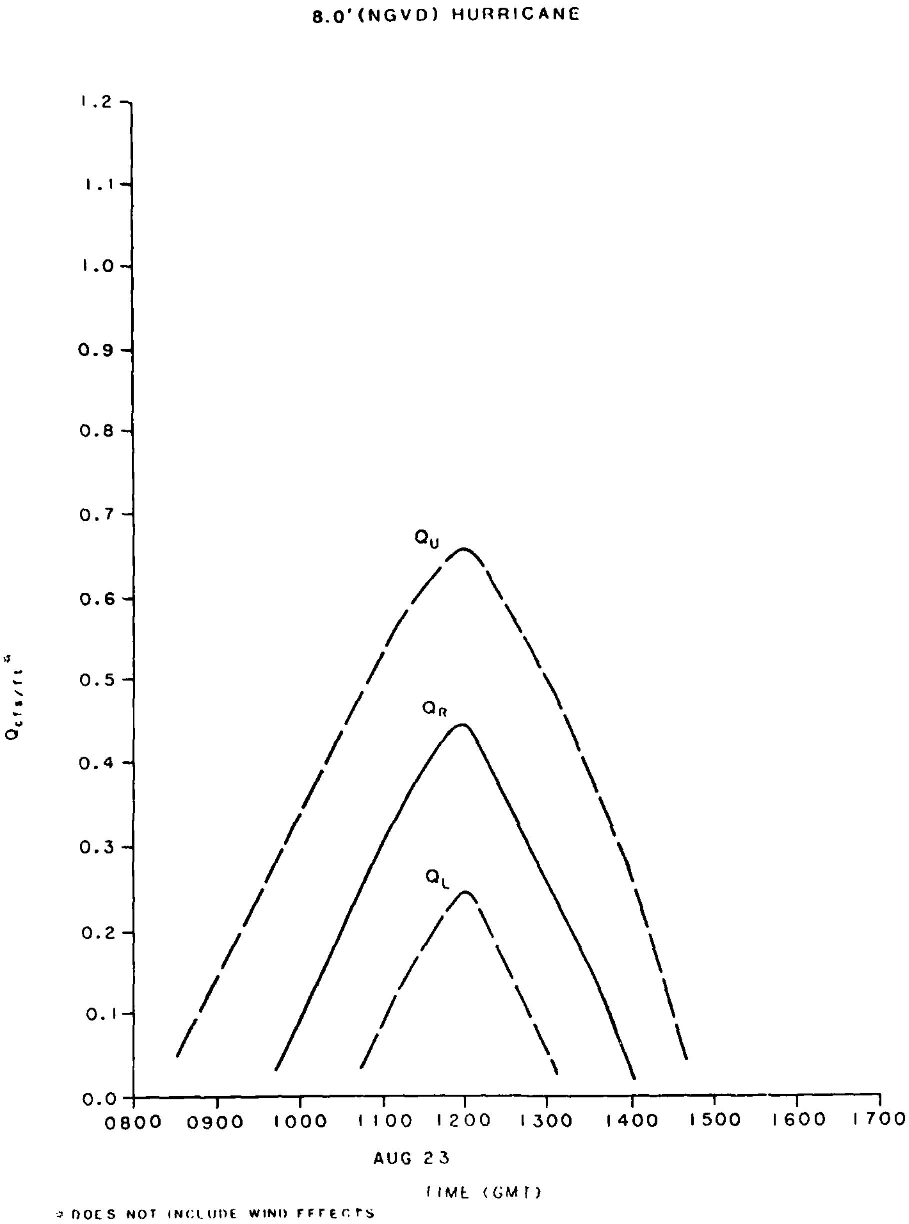


Figure 9. Virginia Beach overtopping hydrograph

and swl associated with this portion of the data is 100 percent gain and +8.0 swl, the design condition. The curve appeared to be more controlled by data generated by low wave heights (i.e., gains) produced at the wave paddle. Use of the RFM curve to predict the peak storm overtopping rates would produce a lower value than the actual model study data for a particular condition. This occurrence could be a function of phenomena such as scaling effect, wave/tank

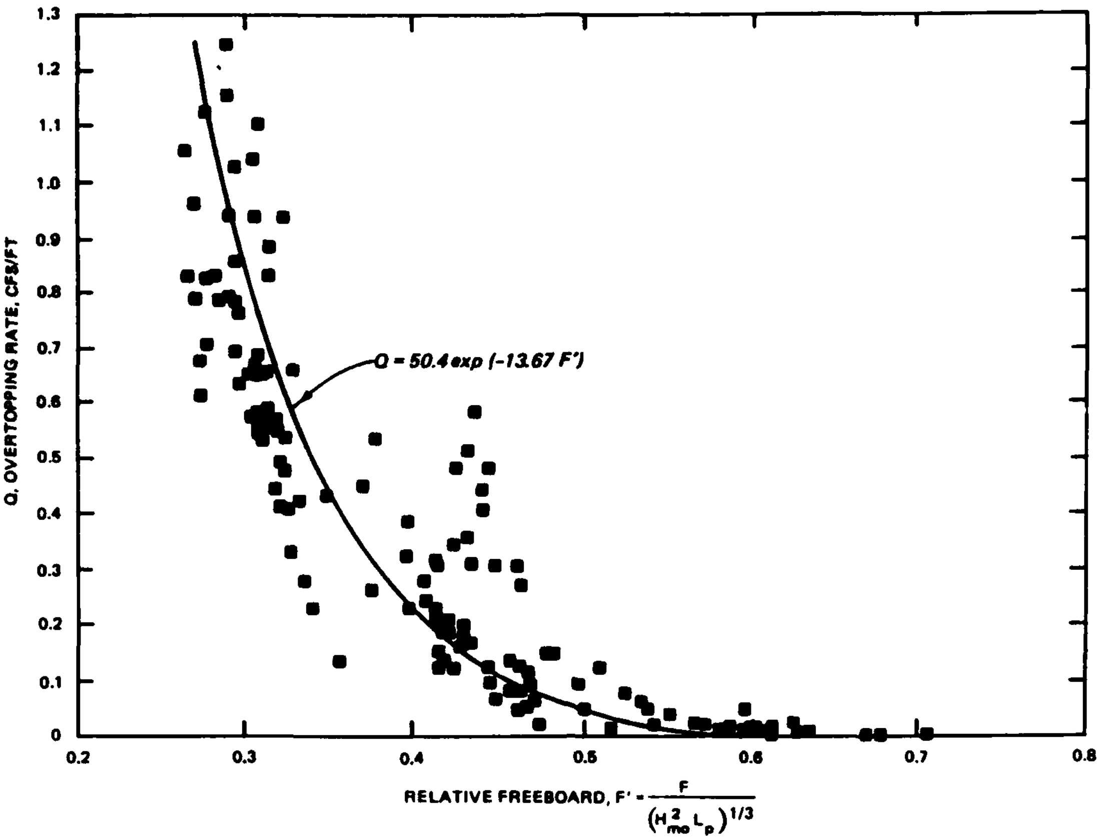


Figure 10. Q versus F' for Phase II seawall

interactions, or various runup effects caused by the complex seawall geometry for the various swl's.

37. The RFM did provide a predictive tool for estimating overtopping rates with the beach elevation at 3.4 ft above NGVD. NAO requested calculations of overtopping rates, using existing model data, for a bottom elevation of 3.4 ft above NGVD in front of the seawall instead of the +1.0 ft elevation which was tested. Because time requirements prohibited modification of the physical model and additional testing, CERC performed an analysis which meshed the RFM and the STHM to compute these overtopping estimates.

38. The RFM was used to estimate an overtopping rate for a shallower water depth based on its exponential curve. This new value for overtopping was compared to an average overtopping measured from the model tests with deeper water levels (bottom elevation of +3.4 NGVD as opposed to +1.0 NGVD). The percent reduction in average overtopping was computed for each of the three design water levels. These values are shown in Table 6. The percent reductions were then multiplied against the physical model results, and the STHM was again applied. Only the hurricane data at 95 and 100 percent gain

Table 6

Overtopping Adjustments*

Swl ft	Berm Elevation ft, above NGVD	Relative Values cfs/ft	Percent Change
9.5	+ 1.0	0.798	51
	+ 3.4	0.409	
8.0	+ 1.0	0.326	31
	+ 3.4	0.102	
7.0	+ 1.0	0.134	16
	+ 3.4	0.021	

* Based on exponential curve averaged for hurricane data at 95 and 100 percent gain.

were used because these represented the design conditions. Results are presented in the following section.

Wind-Induced Overtopping

39. The physical model results do not include the effects of local wind on overtopping rates. Wind effects can cause an increase in wave energy through wind-to-water energy transfers and thus higher runup and overtopping through increased advection of spray over the structure. To establish a relation between wave-induced and wind-induced overtopping, several steps were taken. A search of pertinent literature was conducted but no documented research suitably addressed the computation of additional overtopping due to wind. There were no published procedures which could be used to base a recommendation for adjusting the Virginia Beach model results to include wind overtopping other than the method described in the SPM (1984). However, other attempts were made to determine relative values and perhaps provide insight into wind-induced overtopping.

40. As part of this study, CERC contracted Dr. Donald Resio of Offshore and Coastal Technologies, Inc., who has conducted analytical studies for the oil industry on wind-induced overtopping.* Dr. Resio used video tapes of the

* Unpublished report by Donald T. Resio (1987), "Assessment of Wind Effects on Wave Overtopping of Proposed Virginia Beach Seawall," prepared for USAE Waterways Experiment Station, Vicksburg, MS.

Virginia Beach Phase II model tests to quantitatively evaluate overtopping potential. He recommended using a different correction factor for each swl. These factors are presented in Section VI of this report. The correction factors represented a percent increase in wind-induced overtopping potential for each swl. Dr. Resio observed from the video tapes that at low swl's more potential for wind-induced overtopping occurred than at the higher swl's. As the surge increased, the seawall became inundated causing less wave breaking and thus less wind-induced overtopping.

41. The SPM (1984) recommends another method for estimating wind-induced overtopping. Appendix C provides a sample calculation which is based on the following equation:

$$k' = 1.0 + Wf\ (h - ds/R + 0.1) \sin \theta$$

where

- k' = the wind correction factor
- Wf = a coefficient depending on wind speed
- h = height of the structure crest above the bottom
- ds = depth at the toe of the structure
- R = runup on the structure that would occur if the structure were high enough to prevent overtopping
- θ = structure slope (90 being vertical)

42. The increase in overtopping depends on wind velocity and direction with respect to the axis of the structure and structure slope and height. The equation requires use of empirical relations based on monochromatic or regular waves. A comparison of the wind-induced overtopping rates using Resio's method and those calculated using the SPM (1984) suggests the latter method probably results in conservative predictions. This comparison is provided in Part VI.

43. The effects of wave-induced setup in the model were measured. However, there are no prototype measurements of wind- or wave-induced setup for storms along Virginia Beach which would have allowed verification of the relative scaling of these effects. The SPM recommends a procedure for computing setup based on monochromatic waves which would have limited relevance to the spectral conditions used in model testing. Since the model setup would be more consistent with the other model-generated data used in the study, these

setup values were incorporated into the study results. The setup model data were used in these calculations without modifications (Heimbaugh et al. 1988).

PART VI: RESULTS

44. All results were calculated using overtopping rates from the physical model Phase II tests with the wall configured as shown in Figure 6 (Heimbaugh et al. 1988). Appendix A presents the water levels and overtopping rates from these tests. The tabulated data include three different storm surge levels of 9.5, 8.0, and 7.0 ft above NGVD for both the hurricane and extratropical storm. These data were then used to obtain an overtopping hydrograph necessary for determining flooding potential behind the structure. The STHM described in Section V was employed for these calculations.

45. At the request of NAO, overtopping rates for a beach elevation of +3.4 ft NGVD in front of the seawall were developed using a numeric approach because time constraints did not permit returning to the physical model. These adjusted results are presented following the results of the Phase II model tests.

46. Maximum overtopping rates calculated by the STHM for a +1.0-ft beach elevation are summarized in Table 7. Complete overtopping hydrographs are in Appendix B. The overtopping rates (in cubic feet per second per linear foot of seawall) do not include any contribution due to wind-induced overtopping.

Table 7

Summary of Phase II Overtopping Rates

+1.0-Ft Beach Elevation

Storm Event	Maximum Swl* ft	Overtopping Rate** cfs/ft
Extratropical storm	7.0	0.07
Extratropical storm	7.1	0.10
Extratropical storm†	8.0	0.36
Hurricane	8.0	0.45
Hurricane	8.7	0.67
Hurricane	9.5	0.94

* Swl is in feet referenced to NGVD.

** Overtopping rate in cubic feet per second per linear foot of seawall.

† Not considered a design storm event.

47. Although the swl's are the same for both classes of storms at +8.0 NGVD, the overtopping rate for the hurricane is higher. This occurrence is due to steeper wave conditions which are more characteristic of tropical storms. However, a comparison of the duration of overtopping (Figure 3) shows that overtopping continues for a longer period of time during the extra-tropical storm. During this storm, approximately 9 hr of overtopping and flooding occur as opposed to 5 hr during the hurricane. This increased overtopping duration is due to the length of the storm spanning several tidal cycles compared to the typically faster moving tropical storms. For this reason, not only are the maximum rates important but also the total volume associated with each storm event.

48. As described in section V, using only the storm data at 95 and 100 percent gain, the RFM was used to adjust the shallow-water wave height for the various water depths and to calculate overtopping. Then a change in percent between the overtopping rates for the two bottom elevations was calculated. Table 6 presents this percent reduction in overtopping for each swl which was applied to the actual overtopping rates measured in the model tests. New regression curves were calculated and overtopping hydrographs prepared.

49. The decrease in water depth when the berm elevation was raised to +3.4 ft NGVD significantly reduced both the maximum overtopping rate and the duration of overtopping. Results of the hurricane at an 8.0- and 9.5-ft NGVD surge level are presented in Tables 8 and 9. Only the hurricane data were recalculated for the 3.4-ft elevation because the hurricane represents the maximum design storm with respect to overtopping.

50. Results of the wind-induced overtopping study suggest that overtopping rates computed using the SPM (1984) result in conservative predictions. Table 10 summarizes the contribution due to wind on total overtopping as computed using the SPM procedure and Resio's visual observation correction factor.

51. An earlier method recommended by Resio as a very generalized rule of thumb was used for Roughans Point (Hardy and Crawford 1986) which has a similar freeboard range as was used in the Virginia Beach seawall design. This method reduces the predicted wind effect contribution from the SPM to overtopping by 70 percent for each water level. This reduction would produce values closer to those calculated using Resio's observed potential for

Table 8

1933 Hurricane at +3.4 Ft Beach Elevation, +8.0 Swl

Date	Time*	Swl ft	Swl ft	H_{mo} m	Percent	Q cfs/ft
Aug 23	0800	5.6	5.2	4.6	95	0.0
	0900	6.2	5.7	4.6	95	0.0
	1000	6.8	6.3	4.7	98	0.0
	1100	7.5	6.9	4.8	100	0.0
	1200	8.2	7.5	4.8	100	0.06
	1300	8.7	8.0	4.8	100	0.16
	1400	8.1	7.5	4.8	100	0.05
	1500	7.3	6.7	4.8	100	0.0
	1600	6.2	5.7	4.5	93	0.0

* Greenwich mean time.

Table 9

1933 Hurricane at +3.4 Ft Beach Elevation, +9.5 Swl

Date	Time*	Swl ft	Swl ft	H_{mo} m	Percent	Q cfs/ft
Aug 23	0800	5.6	6.1	4.6	95	0.0
	0900	6.2	6.8	4.6	95	0.0
	1000	6.8	7.4	4.7	98	0.04
	1100	7.5	8.2	4.8	100	0.21
	1200	8.2	9.0	4.8	100	0.37
	1300	8.7	9.5	4.8	100	0.47
	1400	8.1	8.8	4.8	100	0.33
	1500	7.3	8.0	4.8	100	0.16
	1600	6.2	6.8	4.5	93	0.0

* Greenwich mean time.

Table 10

Wind Contribution to Overtopping

Procedure	Q(total) cfs/ft 7.0 Swl	8.0 Swl	9.5 Swl
Q(OT)*	0.148	0.480	0.793
SPM**	0.360	1.070	1.790
Resio†	0.310	0.830	1.050

* Q(total) measured from Phase II model tests (cfs/ft), for a +1 NGVD elevation beach; does not include wind-induced contribution.

** Q(total) = Q(OT) + Q(wind) as calculated using the SPM method.

† Q(total) = Q(OT) × (1 + k') (paragraph 40) where k'= 1.09 for 7.0 swl, 0.73 for 8.0 swl, and 0.33 for 9.5 swl.

overtopping for the +8.0 swl. However, with no data to verify or dispute estimates from any method, there is no factual basis to recommend one type of adjustment over another.

52. As a result of the physical model tests, the stone toe protection was evaluated for its role in reducing overtopping and providing scour protection. Based on observations made during the model tests at lower swl's the riprap appeared to reduce overtopping. However, at the higher swl's the riprap did not reduce overtopping. Actual values were never calculated because model tests were not run without the stone toe protection.

53. The riprap was originally included to control scour at the base of the seawall, thus reducing the risk of structure undermining. However, in light of the proposed structure design, which includes vertical steel sheet pile to a depth sufficient to assure protection of the seawall base in spite of the expected toe scour and the proposed pile support system for the seawall, the stone toe protection is not required. The design and cost of the structure may warrant its removal from the final design.

PART VII: CONCLUSION

54. Using the STHM and a minimum beach elevation of +3.4 ft NGVD, it appears the Phase II seawall design will reduce overtopping rates and volumes to a suitable level as determined by NAO. Calculated values show that with the beach elevation held at +1.0 ft NGVD, overtopping rates increase significantly. However, adjusted values for the shallower beach elevation show that under the design storm events much lower overtopping rates are expected. The lower overtopping rates can be assured only if the beach is well maintained through the life of the project. Since these overtopping rates are for general conditions, there could be localized areas that experience higher or lower overtopping rates due to the two-dimensional nature of the physical model which cannot consider three-dimensional effects such as wave focusing, divergence, or bathymetry variations along the length of the study area. It is also difficult to predict the degree to which wind-induced overtopping will affect the total volume and peak rates during a storm.

55. The most critical time for the project would be after the passage of more than one significant storm between renourishment intervals. The beach would be in an eroded state which makes the seawall more susceptible to storm-induced overtopping. Proper and prompt maintenance of the beach template after erosion events is critical for effective project performance.

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APPENDIX A: PHYSICAL MODEL PHASE II OVERTOPPING RATES

Table A1 presents a summary of Phase II tests. In the column labeled "Storm Type" NE refers to northeasters (extratropical storms), and H refers to hurricanes.

Table A1

Summary of Phase II Tests

Test No.	Store Type	Percent Gain	Over-topping Rate cfs/ft	Swl Prototype ft	Test No.	Store Type	Percent Gain	Over-topping Rate cfs/ft	Swl Prototype ft
240	NE	100	0.966	9.5	204	H	70	0.547	9.5
230	NE	100	0.790	9.5	242	H	70	0.685	9.5
215	NE	100	0.833	9.5	232	H	70	0.666	9.5
220	NE	100	1.127	9.5	166	H	70	0.537	9.5
161	NE	100	0.941	9.5	241	H	60	0.537	9.5
214	NE	90	0.675	9.5	203	H	60	0.447	9.5
219	NE	90	1.105	9.5	165	H	60	0.413	9.5
239	NE	90	0.832	9.5	221	H	60	0.834	9.5
229	NE	90	0.826	9.5	164	H	50	0.332	9.5
160	NE	90	0.612	9.5	163	H	40	0.278	9.5
238	NE	80	0.573	9.5	162	H	30	0.137	9.5
228	NE	80	0.883	9.5	133	NE	100	0.448	8.0
218	NE	80	0.651	9.5	186	NE	100	0.513	8.0
159	NE	80	0.653	9.5	132	NE	90	0.324	8.0
227	NE	70	0.939	9.5	185	NE	90	0.387	8.0
212	NE	70	0.493	9.5	184	NE	80	0.279	8.0
158	NE	70	0.567	9.5	131	NE	80	0.230	8.0
237	NE	70	0.590	9.5	183	NE	70	0.208	8.0
217	NE	70	0.565	9.5	130	NE	70	0.169	8.0
236	NE	60	0.546	9.5	182	NE	60	0.163	8.0
226	NE	60	0.658	9.5	129	NE	60	0.123	8.0
216	NE	60	0.473	9.5	128	NE	50	0.050	8.0
157	NE	60	0.533	9.5	181	NE	50	0.122	8.0
211	NE	60	0.406	9.5	180	NE	40	0.095	8.0
156	NE	50	0.425	9.5	126	NE	30	0.012	8.0
155	NE	40	0.228	9.5	179	NE	30	0.047	8.0
154	NE	30	0.262	9.5	265	H	100	0.585	8.0
245	H	100	0.704	9.5	255	H	100	0.197	8.0
209	H	100	1.040	9.5	250	H	100	0.442	8.0
225	H	100	1.157	9.5	260	H	100	0.480	8.0
235	H	100	1.058	9.5	178	H	95	0.535	8.0
170	H	95	0.787	9.5	153	H	95	0.307	8.0
206	H	90	0.760	9.5	125	H	95	0.346	8.0
168	H	90	0.793	9.5	259	H	90	0.355	8.0
244	H	90	0.856	9.5	249	H	90	0.308	8.0
224	H	90	1.249	9.5	254	H	90	0.405	8.0
234	H	90	0.944	9.5	264	H	90	0.479	8.0
243	H	80	0.780	9.5	177	H	90	0.430	8.0
205	H	80	0.648	9.5	123	H	90	0.308	8.0
233	H	80	0.694	9.5	151	H	90	0.273	8.0
223	H	80	1.029	9.5	253	H	80	0.245	8.0
167	H	80	0.634	9.5	258	H	80	0.230	8.0
222	H	70	0.582	9.5	150	H	80	0.185	8.0

(Continued)

Table A1 (Concluded)

Test No.	Store Type	Percent Gain	Over-topping Rate cfs/ft	Swl Prototype ft
122	H	80	0.182	8.0
248	H	80	0.231	8.0
176	H	80	0.306	8.0
263	H	80	0.315	8.0
252	H	70	0.151	8.0
247	H	70	0.121	8.0
149	H	70	0.095	8.0
257	H	70	0.136	8.0
121	H	70	0.122	8.0
262	H	70	0.209	8.0
175	H	70	0.186	8.0
256	H	60	0.083	8.0
120	H	60	0.067	8.0
174	K	60	0.113	8.0
261	H	60	0.125	8.0
246	H	60	0.082	8.0
251	H	60	0.093	8.0
173	H	50	0.075	8.0
119	H	50	0.046	8.0
118	H	40	0.023	8.0
172	H	40	0.063	8.0
171	H	30	0.039	8.0
194	NE	100	0.149	7.0
141	NE	100	0.137	7.0
140	NE	90	0.077	7.0
193	NE	90	0.094	7.0

Test No.	Store Type	Percent Gain	Over-topping Rate cfs/ft	Swl Prototype ft
139	NE	80	0.045	7.0
192	NE	80	0.048	7.0
191	NE	70	0.023	7.0
138	NE	70	0.019	7.0
190	NE	60	0.009	7.0
137	NE	60	0.008	7.0
189	NE	50	0.001	7.0
136	NE	50	0.004	7.0
135	NE	40	0.004	7.0
188	NE	40	0.011	7.0
134	NE	30	0.002	7.0
187	NE	30	0.004	7.0
202	H	95	0.148	7.0
201	H	90	0.123	7.0
200	H	80	0.083	7.0
146	H	70	0.019	7.0
199	H	70	0.037	7.0
145	H	60	0.015	7.0
148	H	60	0.061	7.0
198	H	60	0.019	7.0
144	H	50	0.008	7.0
197	H	50	0.011	7.0
196	H	40	0.008	7.0
143	H	40	0.005	7.0
195	H	30	0.002	7.0

APPENDIX B: STORM TIME-HISTORY METHOD

1. This appendix contains data used in the regression analysis and subsequent overtopping results from the analysis with the berm at +1.0 NGVD. The first set of tables consists of the Phase II model overtopping values separated by type of storm (H for hurricane and NE for northeaster or extratropical storm) and by percent gain of the wave paddle. Only the 70 percent gain and higher values are presented because at the lower gains no overtopping would have occurred when coupled with design storm surge hydrographs.

2. Following these tables are results of the regression analysis. Results of the regression analysis follow the input values. The numbers following the U( ) and L( ) symbols are upper and lower prediction intervals described in Part V (main text). The percentages enclosed in parentheses are the prediction intervals.

3. Actual regression curves, which present overtopping rates versus swl for each storm type and the various gains form 70 to 100 percent, follow the tabulated data.

4. Final results of the overtopping analysis using the STHM are presented in both tabular and graphic form for each class of storm and percent gain. Generation of the tables and graphs is described in Part V (main text).

NE storm at 100% gain

Linear Regression
B0 =-2.036036   B1 = .312624

Var(B0) =   .078376   Var(B1) =   .001042

1	X = 7.00		Y-Est =	.1523	VAR =	.014148
	L(80)=	.045755	L(90)=	-.015977	L(95)=	-.128974
	U(80)=	.258903	U(90)=	.320635	U(95)=	.433631
2	X = 7.50		Y-Est =	.3086	VAR =	.012730
	L(80)=	.207547	L(90)=	.148990	L(95)=	.041804
	U(80)=	.409734	U(90)=	.468291	U(95)=	.575477
3	X = 8.00		Y-Est =	.4650	VAR =	.011833
	L(80)=	.367485	L(90)=	.311029	L(95)=	.207687
	U(80)=	.562419	U(90)=	.618876	U(95)=	.722218
4	X = 8.50		Y-Est =	.6213	VAR =	.011457
	L(80)=	.525359	L(90)=	.469806	L(95)=	.368121
	U(80)=	.717170	U(90)=	.772722	U(95)=	.874408
5	X = 9.00		Y-Est =	.7776	VAR =	.011602
	L(80)=	.681067	L(90)=	.625165	L(95)=	.522839
	U(80)=	.874085	U(90)=	.929987	U(95)=	1.032313
6	X = 9.50		Y-Est =	.9339	VAR =	.012267
	L(80)=	.834650	L(90)=	.777167	L(95)=	.671948
	U(80)=	1.033126	U(90)=	1.090609	U(95)=	1.195828

Figure B1. Regression analysis results (Sheet 1 of 8)

NE storm at 90% gain

Linear Regression
B0 =-1.968573   B1 = .292247

Var(B0) =   .160750   Var(B1) =   .002136

1	X = 7.00		Y-Est =	.0772	VAR =	.029017
	L(80)=	-.075470	L(90)=	-.163879	L(95)=	-.325705
	U(80)=	.229785	U(90)=	.318194	U(95)=	.480020
2	X = 7.50		Y-Est =	.2233	VAR =	.026109
	L(80)=	.078502	L(90)=	-.005360	L(95)=	-.158864
	U(80)=	.368060	U(90)=	.451922	U(95)=	.605427
3	X = 8.00		Y-Est =	.3694	VAR =	.024270
	L(80)=	.229819	L(90)=	.148965	L(95)=	.000967
	U(80)=	.508990	U(90)=	.589844	U(95)=	.737842
4	X = 8.50		Y-Est =	.5155	VAR =	.023498
	L(80)=	.378179	L(90)=	.298620	L(95)=	.152993
	U(80)=	.652878	U(90)=	.732436	U(95)=	.878063
5	X = 9.00		Y-Est =	.6617	VAR =	.023795
	L(80)=	.523438	L(90)=	.443379	L(95)=	.296835
	U(80)=	.799866	U(90)=	.879925	U(95)=	1.026469
6	X = 9.50		Y-Est =	.8078	VAR =	.025160
	L(80)=	.665653	L(90)=	.583330	L(95)=	.432642
	U(80)=	.949898	U(90)=	1.032221	U(95)=	1.182909

Figure B1. (Sheet 2 of 8)

NE storm at 80% gain

Linear Regression
B0 =-1.814305   B1 =   .262889

Var(B0) =   .079384   Var(B1) =   .001082

1	X = 7.00		Y-Est = .0259		VAR = .013388	
	L(80)=	-.078915	L(90)=	-.140704	L(95)=	-.257222
	U(80)=	.130749	U(90)=	.192537	U(95)=	.309056
2	X = 7.50		Y-Est = .1574		VAR = .012036	
	L(80)=	.057965	L(90)=	-.000620	L(95)=	-.111097
	U(80)=	.256758	U(90)=	.315342	U(95)=	.425819
3	X = 8.00		Y-Est = .2888		VAR = .011225	
	L(80)=	.192818	L(90)=	.136243	L(95)=	.029554
	U(80)=	.384793	U(90)=	.441369	U(95)=	.548057
4	X = 8.50		Y-Est = .4203		VAR = .010954	
	L(80)=	.325426	L(90)=	.269536	L(95)=	.164141
	U(80)=	.515074	U(90)=	.570964	U(95)=	.676359
5	X = 9.00		Y-Est = .5517		VAR = .011225	
	L(80)=	.455707	L(90)=	.399131	L(95)=	.292443
	U(80)=	.647682	U(90)=	.704258	U(95)=	.810946
6	X = 9.50		Y-Est = .6831		VAR = .012036	
	L(80)=	.583742	L(90)=	.525158	L(95)=	.414681
	U(80)=	.782535	U(90)=	.841120	U(95)=	.951597

Figure B1. (Sheet 3 of 8)

NE storm at 70% gain

Linear Regression
B0 =-1.525167   B1 =   .218167

Var(B0) =   .012179   Var(B1) =   .000166

1	X = 7.00	Y-Est =	.0020	VAR =	.002054	
	L(80)=	-.039062	L(90)=	-.063264	L(95)=	-.108903
	U(80)=	.043062	U(90)=	.067264	U(95)=	.112903
2	X = 7.50	Y-Est =	.1111	VAR =	.001847	
	L(80)=	.072151	L(90)=	.049204	L(95)=	.005931
	U(80)=	.150016	U(90)=	.172963	U(95)=	.216236
3	X = 8.00	Y-Est =	.2202	VAR =	.001722	
	L(80)=	.182569	L(90)=	.160409	L(95)=	.118620
	U(80)=	.257764	U(90)=	.279924	U(95)=	.321713
4	X = 8.50	Y-Est =	.3293	VAR =	.001681	
	L(80)=	.292108	L(90)=	.270217	L(95)=	.228935
	U(80)=	.366392	U(90)=	.388283	U(95)=	.429565
5	X = 9.00	Y-Est =	.4383	VAR =	.001722	
	L(80)=	.400736	L(90)=	.378576	L(95)=	.336787
	U(80)=	.475931	U(90)=	.498091	U(95)=	.539880
6	X = 9.50	Y-Est =	.5474	VAR =	.001847	
	L(80)=	.508484	L(90)=	.485537	L(95)=	.442264
	U(80)=	.586349	U(90)=	.609296	U(95)=	.652569

Figure B1. (Sheet 4 of 8)

Hurricane at 100% and 95% gain

Linear Regression
BO =-2.166436   B1 = .327671

Var(BO) = .175615   Var(B1) = .002383

1	X =	7.00	Y-Est =	.1273	VAR =	.028202
	L(80)=	-.020356	L(90)=	-.103147	L(95)=	-.246898
	U(80)=	.274871	U(90)=	.357663	U(95)=	.501414
2	X =	7.50	Y-Est =	.2911	VAR =	.025123
	L(80)=	.151769	L(90)=	.073626	L(95)=	-.062053
	U(80)=	.430418	U(90)=	.508560	U(95)=	.644239
3	X =	8.00	Y-Est =	.4549	VAR =	.023237
	L(80)=	.320937	L(90)=	.245787	L(95)=	.115302
	U(80)=	.588919	U(90)=	.664070	U(95)=	.794555
4	X =	8.50	Y-Est =	.6188	VAR =	.022542
	L(80)=	.486792	L(90)=	.412774	L(95)=	.284256
	U(80)=	.750735	U(90)=	.824753	U(95)=	.953272
5	X =	9.00	Y-Est =	.7826	VAR =	.023038
	L(80)=	.649182	L(90)=	.574353	L(95)=	.444427
	U(80)=	.916016	U(90)=	.990845	U(95)=	1.120771
6	X =	9.50	Y-Est =	.9464	VAR =	.024726
	L(80)=	.808215	L(90)=	.730694	L(95)=	.596091
	U(80)=	1.084653	U(90)=	1.162175	U(95)=	1.296777

Figure B1. (Sheet 5 of 8)

Hurricane at 90% gain

Linear Regression
B0 =-2.040595   B1 = .302190

Var(B0) = .051538   Var(B1) = .000721

1	X = 7.00	Y-Est =	.0747	VAR =	.007629	
	L(80)= -.002042	L(90)=	-.045101	L(95)=	-.119865	
	U(80)= .151505	U(90)=	.194564	U(95)=	.269328	
2	X = 7.50	Y-Est =	.2258	VAR =	.006788	
	L(80)= .153408	L(90)=	.112792	L(95)=	.042268	
	U(80)= .298244	U(90)=	.338861	U(95)=	.409384	
3	X = 8.00	Y-Est =	.3769	VAR =	.006307	
	L(80)= .307113	L(90)=	.267961	L(95)=	.199980	
	U(80)= .446728	U(90)=	.485881	U(95)=	.553862	
4	X = 8.50	Y-Est =	.5280	VAR =	.006187	
	L(80)= .458876	L(90)=	.420098	L(95)=	.352768	
	U(80)= .597155	U(90)=	.635933	U(95)=	.703263	
5	X = 9.00	Y-Est =	.6791	VAR =	.006427	
	L(80)= .608641	L(90)=	.569118	L(95)=	.500492	
	U(80)= .749580	U(90)=	.789103	U(95)=	.857728	
6	X = 9.50	Y-Est =	.8302	VAR =	.007028	
	L(80)= .756517	L(90)=	.715187	L(95)=	.643427	
	U(80)= .903894	U(90)=	.945223	U(95)=	1.016984	

Figure B1. (Sheet 6 of 8)

Hurricane at 80% gain

Linear Regression
B0 =-2.229348   B1 = .312611

Var(B0) =   .106769   Var(B1) =   .001464

1	X = 7.00		Y-Est =	-.0411	VAR =	.017480
	L(80)=	-.156890	L(90)=	-.221278	L(95)=	-.332073
	U(80)=	.074749	U(90)=	.139137	U(95)=	.249932
2	X = 7.50		Y-Est =	.1152	VAR =	.015651
	L(80)=	.005645	L(90)=	-.055280	L(95)=	-.160117
	U(80)=	.224825	U(90)=	.285751	U(95)=	.390587
3	X = 8.00		Y-Est =	.2715	VAR =	.014553
	L(80)=	.165864	L(90)=	.107114	L(95)=	.006022
	U(80)=	.377217	U(90)=	.435967	U(95)=	.537060
4	X = 8.50		Y-Est =	.4278	VAR =	.014187
	L(80)=	.323506	L(90)=	.265500	L(95)=	.165687
	U(80)=	.532186	U(90)=	.590192	U(95)=	.690006
5	X = 9.00		Y-Est =	.5842	VAR =	.014553
	L(80)=	.478475	L(90)=	.419725	L(95)=	.318633
	U(80)=	.689829	U(90)=	.748578	U(95)=	.849671
6	X = 9.50		Y-Est =	.7405	VAR =	.015651
	L(80)=	.630867	L(90)=	.569942	L(95)=	.465106
	U(80)=	.850048	U(90)=	.910973	U(95)=	1.015809

Figure B1. (Sheet 7 of 8)

Hurricane at 70% gain

Linear Regression
B0 =-1.873379   B1 = .258045

Var(B0) =   .034213   Var(B1) =   .000480

1	X = 7.00	Y-Est = -.0671		VAR = .006638		
	L(80)=	-.138192	L(90)=	-.177545	L(95)=	-.244600
	U(80)=	.004066	U(90)=	.043419	U(95)=	.110474
2	X = 7.50	Y-Est = .0620		VAR = .006089		
	L(80)=	-.006165	L(90)=	-.043856	L(95)=	-.108079
	U(80)=	.130084	U(90)=	.167775	U(95)=	.231998
3	X = 8.00	Y-Est = .1910		VAR = .005781		
	L(80)=	.124607	L(90)=	.087884	L(95)=	.025310
	U(80)=	.257357	U(90)=	.294080	U(95)=	.356654
4	X = 8.50	Y-Est = .3200		VAR = .005712		
	L(80)=	.254025	L(90)=	.217520	L(95)=	.155319
	U(80)=	.385985	U(90)=	.422489	U(95)=	.484690
5	X = 9.00	Y-Est = .4490		VAR = .005884		
	L(80)=	.382064	L(90)=	.345015	L(95)=	.281887
	U(80)=	.515991	U(90)=	.553039	U(95)=	.616167
6	X = 9.50	Y-Est = .5780		VAR = .006295		
	L(80)=	.508783	L(90)=	.470461	L(95)=	.405161
	U(80)=	.647316	U(90)=	.685639	U(95)=	.750939

Figure B1. (Sheet 8 of 8)

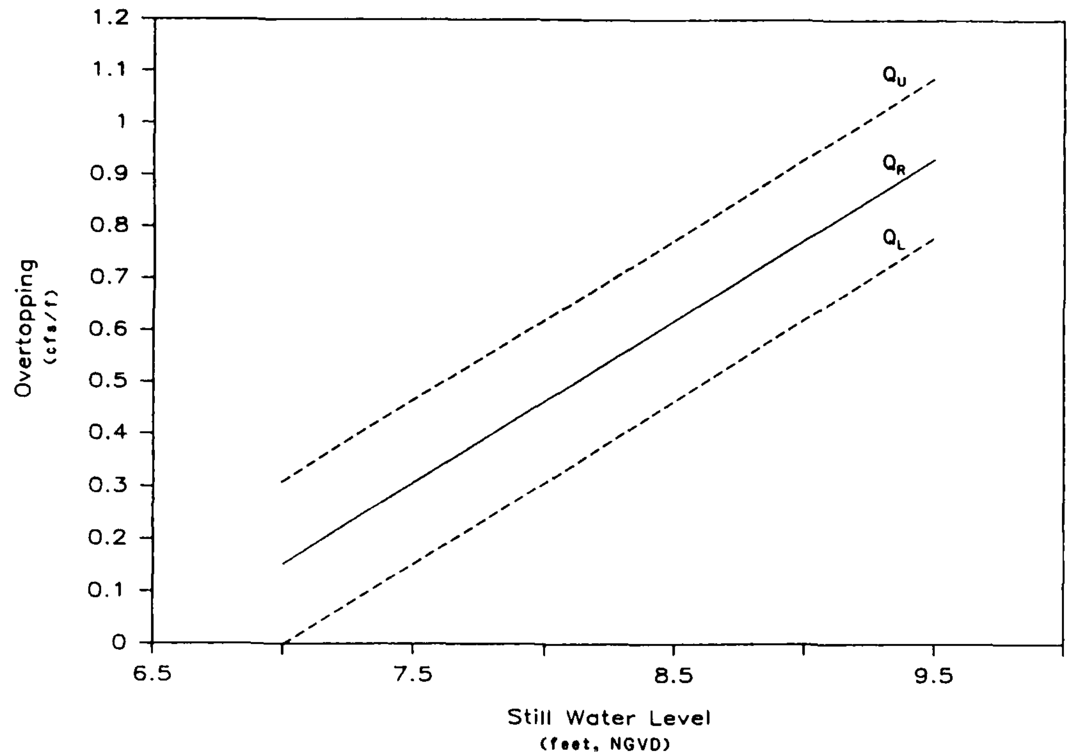

Figure B2. Swl versus overtopping rate, extratropical storm, 100% gain

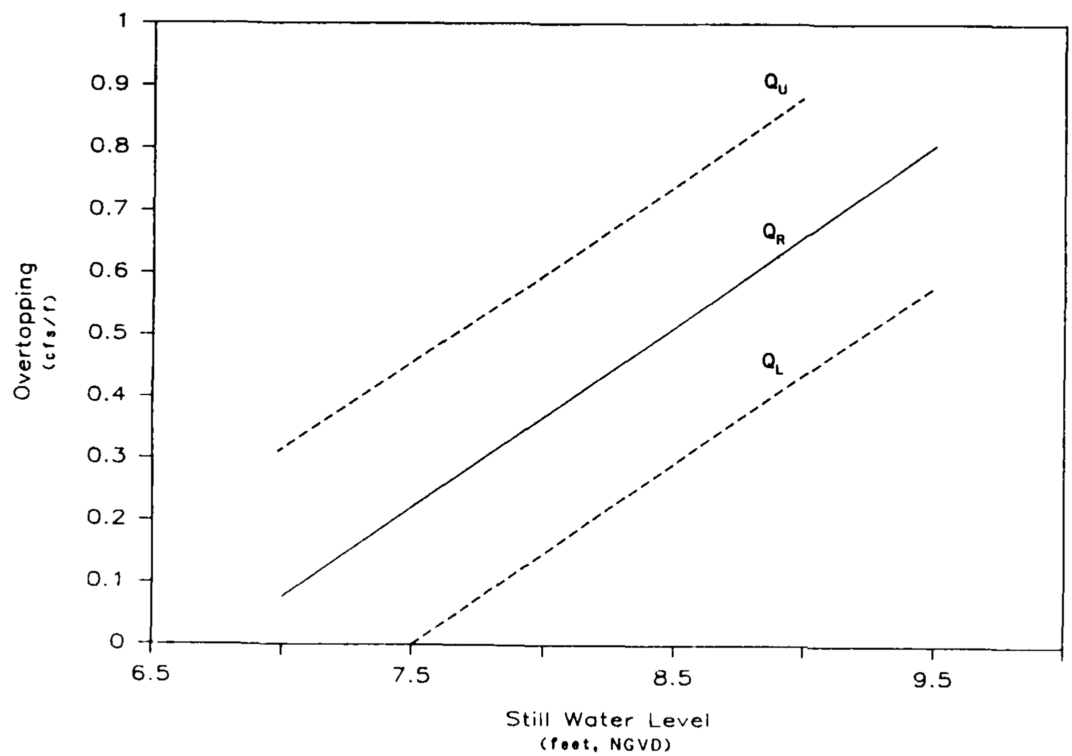

Figure B3. Swl versus overtopping rate, extratropical storm, 90% gain

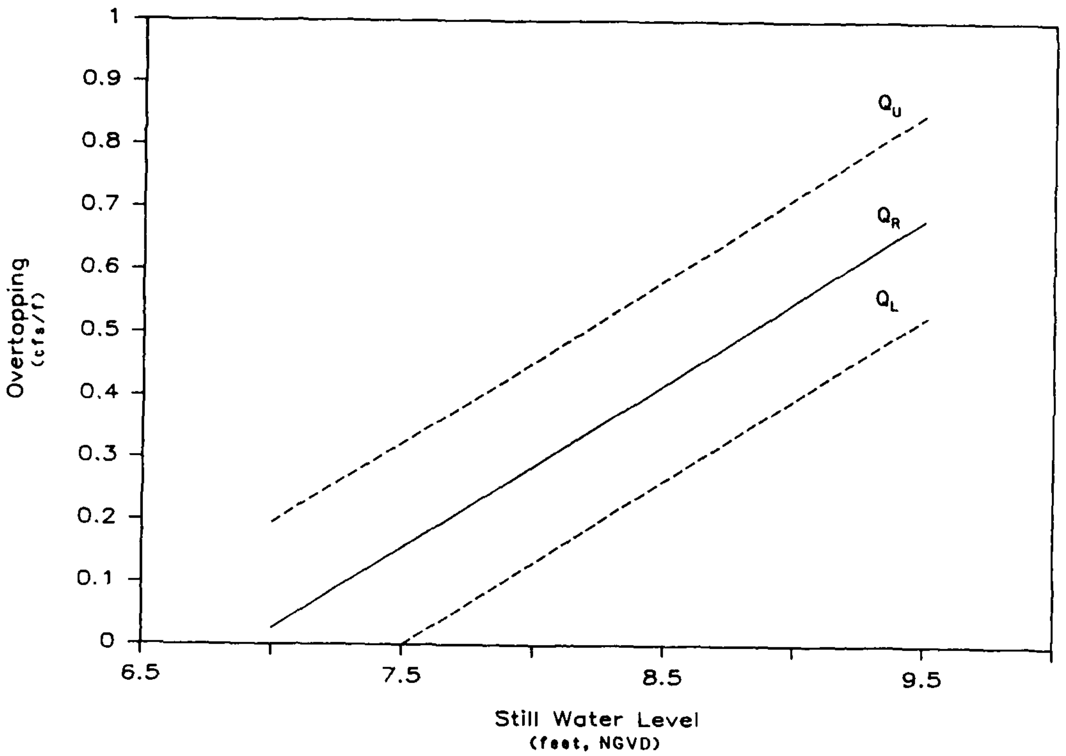

Figure B4. Swl versus overtopping rate, extratropical storm, 80% gain

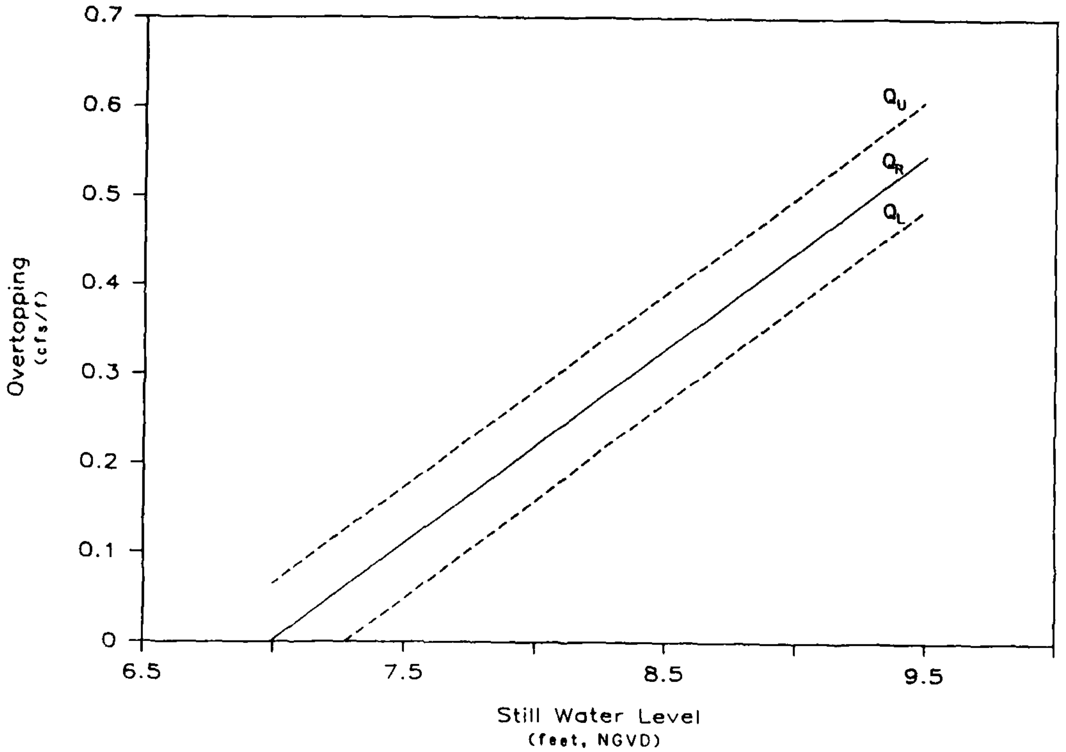

Figure B5. Swl versus overtopping rate, extratropical storm, 70% gain

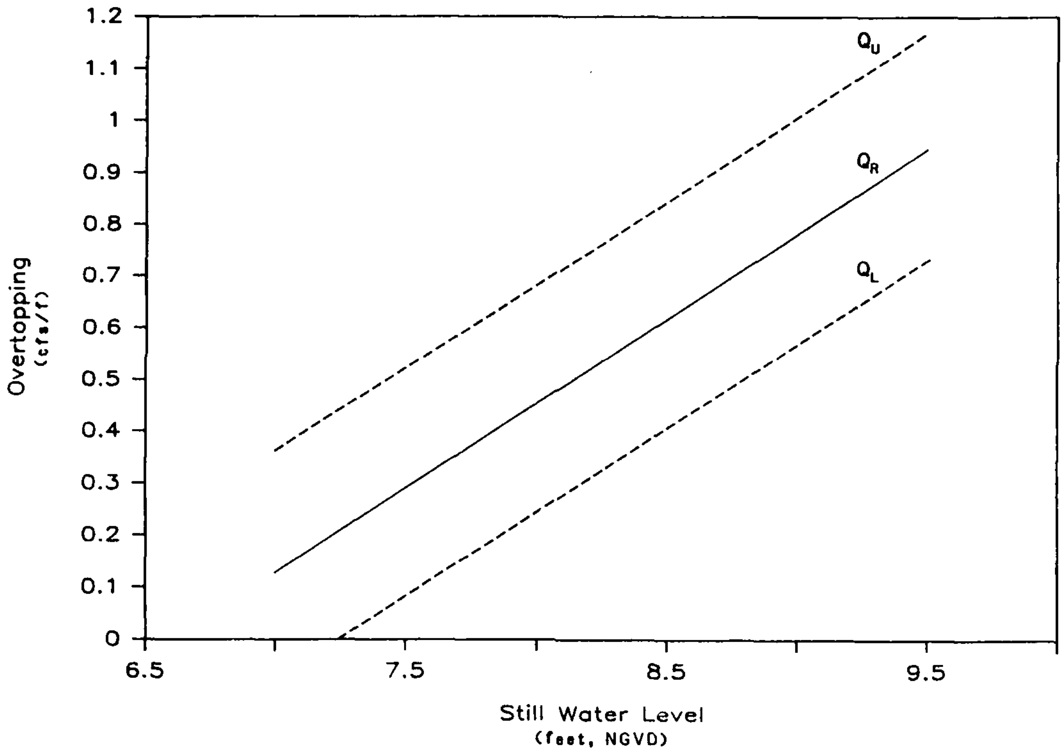

Figure B6. Swl versus overtopping rate, hurricane, 100% and 95% gain

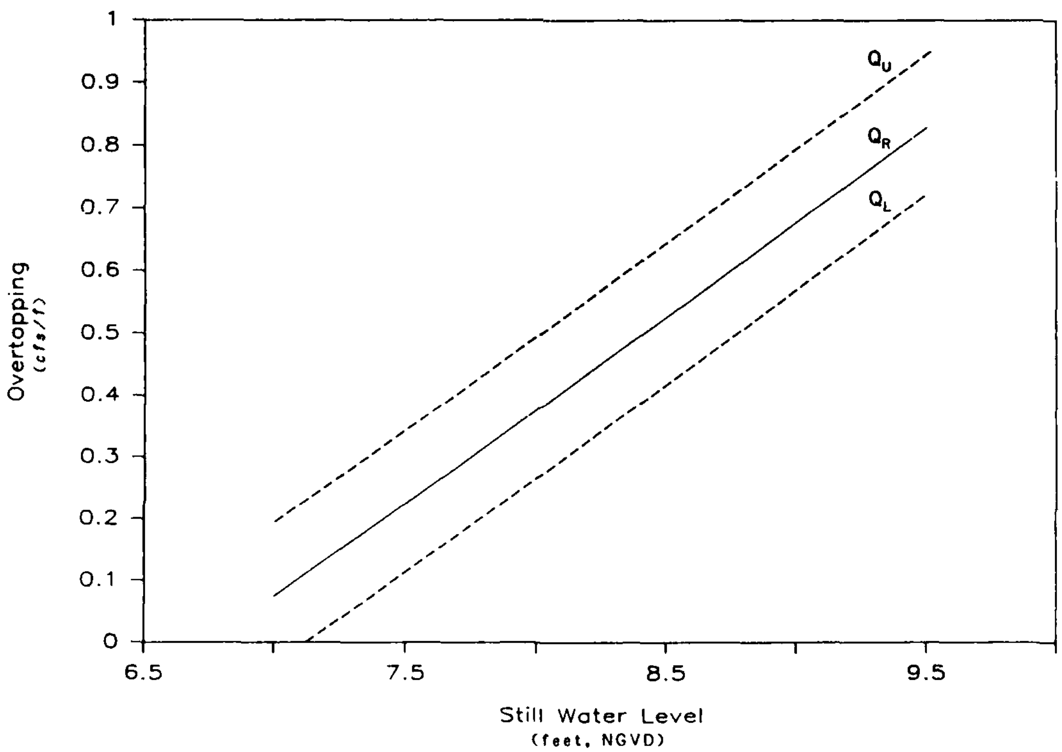

Figure B7. Swl versus overtopping rate, hurricane, 90% gain

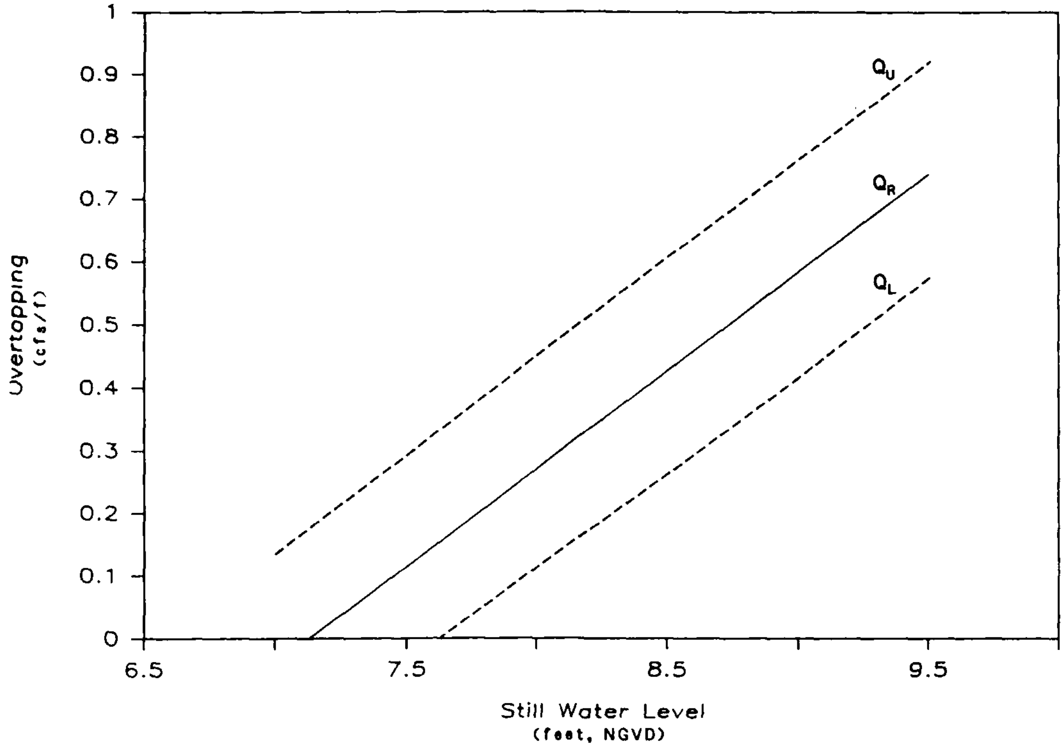

Figure B8. Swl versus overtopping rate, hurricane, 80% gain

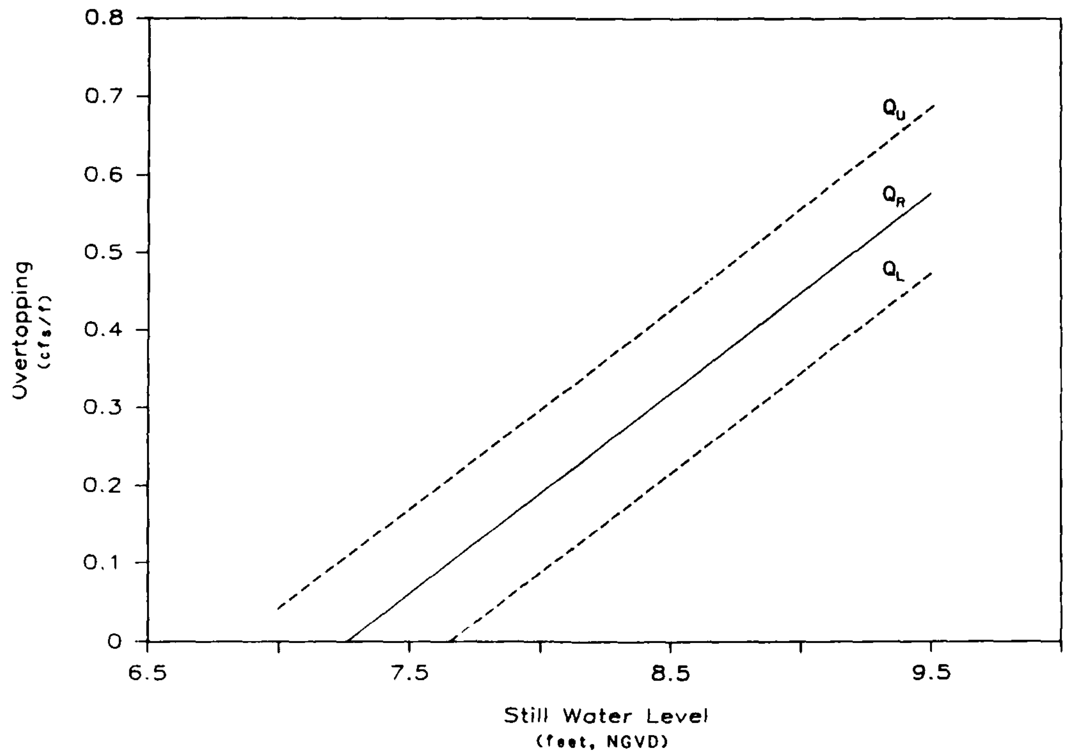

Figure B9. Swl versus overtopping rate, hurricane, 70% gain

Table B1

Extratropical Storm 1962, 7.0-ft Maximum Surge

Date	Time*	Still Water Level ft	Still Water Level** ft	H_{mo}† m	Percent	Q_R cf/s/ft	Q_U cf/s/ft	Q_L cf/s/ft
Mar 6	1700	0.3				0	0	0
	1800	-0.2						
	1900	-0.3						
	2000	0.0	N/A	N/A	N/A			
	2100	1.5						
	2200	2.7						
	2300	4.0						
Mar 7	0000	4.9	5.12	2.3	0.55			
	0100	5.2	5.43	2.45	0.59			
	0200	4.7	4.91	2.61	0.63			
	0300	4.2	4.39	2.85	0.69			
	0400	3.6	3.76	3.00	0.72			
	0500	3.0	3.13	3.11	0.75			
	0600	2.6	2.72	3.25	0.78			
	0700	2.2	2.30	3.35	0.81			
	0800	2.6	2.72	3.45	0.83			
	0900	4.0	4.18	3.55	0.86			
	1000	5.0	5.22	3.55	0.86			
	1100	5.7	5.96	3.55	0.86			
	1200	6.3	6.58	3.60	0.87	0	0.2	0
	1300	6.7	7.0	3.62	0.87	0.07	0.31	0
	1400	6.7	7.0	3.72	0.90	0.07	0.31	0
	1500	6.0	6.27	3.76	0.91	0	0.10	0
	1600	5.4	5.64	3.80	0.92			

* Greenwich mean time.
** Scaled.
† H_{mo} = 4.15 m = 13.6 ft.

Table B2

Extratropical Storm 1962, 7.1-ft Storm Surge

Date	Time*	Still Water Level ft	Still Water Level** ft	H_{mo}† m	Percent	Q_R cf/s/ft	Q_U cf/s/ft	Q_L cf/s/ft
Mar 6	1700	0.3				0	0	0
	1800	-0.2						
	1900	-0.3						
	2000	0.0						
	2100	1.5	N/A	N/A				
	2200	2.7						
	2300	4.0						
Mar 7	0000	4.9	5.19	2.3				
	0100	5.2	5.51	2.45	N/A			
	0200	4.7		2.61				
	0300	4.2		2.85				
	0400	3.6		3.00				
	0500	3.0		3.11				
	0600	2.6	N/A	3.25				
	0700	2.2		3.35				
	0800	2.6		3.45				
	0900	4.0		3.55				
	1000	5.0	5.30	3.55	0.86			
	1100	5.7	6.04	3.55	0.86	0	0.05	0
	1200	6.3	6.68	3.60	0.87	0	0.22	0
	1300	6.7	7.10	3.62	0.87	0.10	0.35	0
	1400	6.7	7.10	3.72	0.90	0.10	0.35	0
	1500	6.0	6.36	3.76	0.91	0	0.14	0
	1600	5.4	5.72	3.80	0.92			

(Continued)

* Greenwich mean time.

** Scaled.

† H_{mo} = 4.15 m = 13.6 ft.

Table B2
(Concluded)

Date	Time*	Still Water Level ft	Still Water Level** ft	H_{mo}† m	Percent	Q_R cf/s/ft	Q_U cf/s/ft	Q_L cf/s/ft
Mar 7	1700	4.4		3.80		0	0	0
	1800	3.8		3.82				
	1900	3.3		3.82				
	2000	3.2	N/A	3.88	N/A			
	2100	3.5		3.90				
	2200	4.1		3.91				
	2300	5.0	5.30	4.00	0.96			
Mar 8	0000	5.6	5.93	4.02	0.97	0	0.0	0
	0100	5.9	6.25	4.02	0.97	0	0.01	0
	0200	6.1	6.46	4.05	0.98	0	0.16	0
	0300	5.9	6.25	4.12	0.99	0	0.01	0
	0400	5.0	5.30	4.11	0.99	0	0	0
	0500	4.1		4.14				
	0600	3.3		4.08				
	0700	2.3		4.00				
	0800	1.6		3.92				
	0900	1.6		3.88				
	1000	2.0		3.82	N/A			
	1100	3.1	N/A	3.77				
	1200	4.1		3.70				
	1300	4.5		3.63				
	1400	4.8		3.60				
	1500	4.7		3.54				
	1600	4.2		3.50				

Table B3

Extratropical Storm 1962, 8.0-ft Storm Surge

Date	Time*	Still Water Level ft	Still Water Level** ft	H_{mo}† m	Percent	Q_R cf/s/ft	Q_U cf/s/ft	Q_L cf/s/ft
Mar 6	1700	0.3				0	0	0
	1800	-0.2						
	1900	-0.3						
	2000	0.0	N/A	N/A				
	2100	1.5						
	2200	2.7						
	2300	4.0						
Mar 7	0000	4.9	5.85	2.3				
	0100	5.2	6.21	2.45	N/A			
	0200	4.7	5.6	2.62				
	0300	4.2		2.85				
	0400	3.6		3.00				
	0500	3.0		3.11				
	0600	2.6	N/A	3.25				
	0700	2.2		3.35				
	0800	2.6		3.45				
	0900	4.0	4.78	3.55				
	1000	5.0	5.97	3.55	0.86	0	0.03	0
	1100	5.7	6.81	3.55	0.86	0.01	0.25	0
	1200	6.3	7.52	3.60	0.87	0.22	0.45	0.0
	1300	6.7	8.0	3.62	0.87	0.36	0.60	0.14
	1400	6.7	8.0	3.72	0.90	0.36	0.60	0.14
	1500	6.0	7.16	3.76	0.91	0.13	0.37	0
	1600	5.4	6.45	3.80	0.92	0	0.15	0

(Continued)

* Greenwich mean time.

** Scaled.

† H_{mo} = 4.15 m = 13.6 ft.

Table B3
(Concluded)

Date	Time*	Still Water Level ft	Still Water Level** ft	H_{mo}† m	Percent	Q_R cf/s/ft	Q_U cf/s/ft	Q_L cf/s/ft
Mar 7	1700	4.4	5.25	3.80		0	0	0
	1800	3.8		3.82				
	1900	3.3		3.82				
	2000	3.2	N/A	3.88	N/A			
	2100	3.5		3.90				
	2200	4.1	4.90	3.91				
	2300	5.0	5.97	4.00	0.96	0	0.0	0
Mar 8	0000	5.6	6.69	4.02	0.97	0.06	0.23	0
	0100	5.9	7.04	4.02	0.97	0.15	0.32	0
	0200	6.1	7.28	4.05	0.98	0.25	0.40	0.09
	0300	5.9	7.04	4.12	0.99	0.17	0.34	0.0
	0400	5.0	5.97	4.11	0.99	0	0.0	0
	0500	4.1	2.23	4.14		0	0	0
	0600	3.3		4.08				
	0700	2.3		4.00				
	0800	1.6		3.92				
	0900	1.6		3.88				
	1000	2.0		3.82				
	1100	3.1	N/A	3.77	N/A			
	1200	4.1		3.70				
	1300	4.5		3.63				
	1400	4.8		3.60				
	1500	4.7		3.54				
	1600	4.2		3.50				

Table B4

Hurricane 1933, 8.0-ft Surge

Date	Time*	Still Water Level ft	Still Water Level** ft	H_{mo}† m	Percent	Q_R cf/s/ft	Q_U cf/s/ft	Q_L cf/s/ft
Aug 22	1900	0.50		3.3	0.68	0	0	0
	2000	0.80		3.4	0.71			
	2100	1.10		3.6	0.75			
	2200	1.60		3.7	0.77			
	2300	2.40		3.8	0.79			
Aug 23	0000	3.30		3.9	0.81			
	0100	3.80	N/A	4.0	0.83			
	0200	4.20		4.2	0.87			
	0300	4.30		4.3	0.89			
	0400	4.30		4.4	0.91			
	0500	4.40		4.4	0.91			
	0600	4.60		4.4	0.91			
	0700	5.00		4.5	0.93			
	0800	5.60	5.15	4.6	0.95			
	0900	6.20	5.7	4.6	0.95			
	1000	6.80	6.25	4.7	0.98	0	0.14	0
	1100	7.50	6.90	4.8	1.00	0.09	0.33	0
	1200	8.20	7.54	4.8	1.00	0.30	0.53	0.08
	1300	8.70	8.00	4.8	1.00	0.45	0.66	0.25
	1400	8.10	7.45	4.9	1.00	0.27	0.50	0.05
	1500	7.30	6.71	4.8	1.00	0.03	0.27	0
	1600	6.20	5.7	4.5	0.93	0	0	0
	1700	4.30		4.1	0.85			
	1800	2.80						

* Greenwich mean time.

** Scaled.

† H_{mo} = 4.82 m = 15.8 ft.

Table B5

Hurricane 1933, 8.7-ft Surge

Date	Time*	Still Water Level ft	Still Water Level** ft	H_{mo}† m	Percent	Q_R cf/s/ft	Q_U cf/s/ft	Q_L cf/s/ft
Aug 22	1900	0.50		3.3	0.68	0	0	0
	2000	0.80		3.4	0.71			
	2100	1.10		3.6	0.75			
	2200	1.60		3.7	0.77			
	2300	2.40		3.8	0.79			
Aug 23	0000	3.30		3.9	0.81			
	0100	3.80	N/A	4.0	0.83			
	0200	4.20		4.2	0.87			
	0300	4.30		4.3	0.89			
	0400	4.30		4.4	0.91			
	0500	4.40		4.4	0.91			
	0600	4.60		4.4	0.91			
	0700	5.00		4.5	0.93			
	0800	5.60		4.6	0.95			
	0900	6.20		4.6	0.95	0	0.12	0
	1000	6.80		4.7	0.98	0.06	0.30	0
	1100	7.50		4.8	1.00	0.29	0.51	0.7
	1200	8.20		4.8	1.00	0.50	0.73	0.30
	1300	8.70	N/A	4.8	1.00	0.67	0.86	0.48
	1400	8.10		4.9	1.00	0.47	0.69	0.27
	1500	7.30		4.8	1.00	0.22	0.45	0.0
	1600	6.20		4.5	0.93	0	0.12	0
	1700	4.30		4.1	0.85			
	1800	2.80						

* Greenwich mean time.
** Scaled.
† H_{mo} = 4.82 m = 15.8 ft.

Table B6

Hurricane 1933, 9.5-ft Surge

Date	Time*	Still Water Level ft	Still Water Level** ft	H_{mo}† m	Percent	Q_R cf/s/ft	Q_U cf/s/ft	Q_L cf/s/ft
Aug 22	1900	0.50		3.3	0.68	0	0	0
	2000	0.80		3.4	0.71			
	2100	1.10		3.6	0.75			
	2200	1.60		3.7	0.77			
	2300	2.40		3.8	0.79			
Aug 23	0000	3.30	N/A	3.9	0.81			
	0100	3.80		4.0	0.83			
	0200	4.20		4.2	0.87			
	0300	4.30		4.3	0.89			
	0400	4.30		4.4	0.91			
	0500	4.40	4.80	4.4	0.91			
	0600	4.60	5.02	4.4	0.91			
	0700	5.00	5.46	4.5	0.93			
	0800	5.60	6.11	4.6	0.95	0	0.10	0
	0900	6.20	6.77	4.6	0.95	0.05	0.30	0
	1000	6.80	7.43	4.7	0.98	0.25	0.49	0.05
	1100	7.50	8.19	4.8	1.00	0.51	0.73	0.30
	1200	8.20	8.95	4.8	1.00	0.75	0.95	0.55
	1300	8.70	9.50	4.8	1.00	0.94	1.10	0.75
	1400	8.10	8.84	4.9	1.00	0.72	0.91	0.53
	1500	7.30	7.97	4.8	1.00	0.45	0.66	0.24
	1600	6.20	6.77	4.5	0.93	0.05	0.30	0
	1700	4.30	4.7	4.1	0.85	0	0	0
	1800	2.80						

* Greenwich mean time.

** Scaled.

† H_{mo} = 4.82 m = 15.8 ft.

APPENDIX C: WIND-INDUCED OVERTOPPING EVALUATION

1. A sample calculation of wind-induced overtopping, using the method described in the SPM (1984), is presented in this appendix. No adaptations were made to include wind-induced setup in these calculations. Following the example is a summary of total estimated maximum overtopping rates for the Virginia Beach seawall design as modeled in the Phase II model tests. These estimates include both wave-induced and wind-induced overtopping. The summary presents results for the cases with the +1.0 ft NGVD elevation and the +3.4 ft elevation.

2. To estimate wind-induced overtopping (as described in Chapter 7 of the SPM) runup on the seawall must be calculated first. For this example a storm surge of +8.0 ft above NGVD will be used. To calculate runup the ratios of H'_o/gT^2 and d_s/H'_o must be determined for use with Figure 7-18 in the SPM, where H'_o is the deepwater wave height, g is gravity, T is the wave period of 13.7 sec, and d_s is the depth from the swl to the toe of the structure, in this case 7.0 ft (8.0 - 1.0 = 7.0).

3. To determine the deepwater wave height the ratio of breaking wave height to deepwater wave height (H_b/H'_o) is evaluated. Breaking wave height H_b is found by calculating d_s/gT^2 and entering Figure 7-4 using a slope of 0.05 (1 on 20). This value d_s/gT^2 is 0.00116 and yields a value of H_b/d_s of 1.35, or H_b equals 9.5 ft.

4. With H_b known, another ratio, d_s/L, occurs where L is the deepwater wave length equal to $5.12T^2$ and where d_s/L equals 0.0073. From Table C1 in Appendix C of the SPM the ratio of H/H'_o equals to 1.546 is found. This allows for the original ratios of H'_o/gT^2 and d_s/H'_o to be determined as follows:

$$\frac{H'_o}{gT^2} = \frac{6.1}{32.2(13.7)^2} = 0.0010$$

and

$$\frac{d_s}{H'_o} = \frac{7.0}{6.1} = 1.148$$

5. Entering Figure 7-18, the ratio of runup to deepwater wave height is found to be 1.53 or runup of 9.3 ft. To adjust for scale effects, 9.3 is multiplied by 1.21, based on the SPM, for a total runup of 11.3 ft.

6. To include wind effects on overtopping, Equation 7-12 of the SPM is used as follows:

$$k' = 1.0 + Wf \frac{h - d_s}{R} + 0.1 \sin 0$$

where

k' = wind correction factor

Wf = coefficient depending on wind speed

h = height of the structure crest above the bottom

d_s = depth at the toe of the structure

R = runup on the structure that would occur if the structure were high enough to prevent overtopping

0 = structure slope (90 being vertical)

7. For this example, a wind speed of 50 mph is used; therefore, based on the SPM, Wf equals 1.5. The value of $h - d_s/R$ equals 0.68 or

$$k' = 1.0 + 1.5(0.68 + 0.1) \sin \theta$$

$$= 1.0 + 1.17 \sin \theta$$

$$\text{for } \theta = 90, \sin \theta = 1.0$$

$$k' = 2.17$$

8. A value of $k' = 2.17$ represents a 117 percent increase in overtopping due to the contribution of wind because total overtopping equals Q(wave-induced) × k'. The following table summarizes the wave-induced and wind-induced maximum overtopping rates based on the Phase II model tests with eroded berm elevations of +1.0 ft and +3.4 ft above NGVD.

Total Overtopping Rates Including Both

Wind- and Wave-Induced Overtopping

	Eroded Berm Elevation		
	+1.0 NGVD Q, cfs/f		+3.4 NGVD Q, cfs/s
	SPM		
7.0 swl	0.360		NA*
8.0 swl	1.070	0.338	
9.5 swl	1.790	1.032	
	Resio		
7.0 swl	0.310		NA
8.0 swl	0.830	0.117	
9.5 swl	1.050	0.155	

* Overtopping calculations for the +7.0 ft NGVD elevation were not calculated because of their expected low values.